Feasibility of Multi Storey Post-Tensioned Timber Buildings: Detailing, Cost and Construction

A thesis submitted in partial fulfilment of the requirements for the
Degree of Master of Engineering in Civil Engineering
at the University of Canterbury

By

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Abstract

This thesis describes a feasibility study into the use of a new method of timber construction developed in New Zealand. This new method combines the use of an engineered wood product (Laminated Veneer Lumber) and post-tensioned ductile connections. Three case study buildings are presented in concrete, steel and timber all representing current design and construction practice. A fourth building, referred to as the “timber plus” structure, is also considered with the addition of timber architectural components.

The case study timber building consists of two lateral resisting systems. In one direction post-tensioned LVL moment resisting frames are used, with post-tensioned cantilever walls in the orthogonal direction. Timber-concrete composite floor units are also used.

The method of design and detailing of the timber building is shown with member sizes being found to be comparable to that of the concrete structure. Sub-assembly testing is performed on some key connections with excellent results. Construction time is evaluated and compared to the concrete structure with similar construction times being achieved. Finally the costs of the case study buildings are calculated and compared. The costing found the four options to be similar in price with the Timber and Timber plus buildings showing only a 6% and 11% increase in total cost respectively.

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Table of Contents

List of Figures

List of Tables

1 Introduction

Timber is one of the most ancient building materials in the world. Multi storey timber buildings date back for thousands of years. The 5 storey, 57m high To-ji pagoda in Kyoto Japan was constructed in 1695 and is to this day the highest timber building in Japan. The older and taller Sakyamuni pagoda (Figure 1.1) in Yingxian province, China was constructed in 1056 and stands at 67.3m, it is the tallest ancient timber structure in the world.

Figure 1.1: Sakyamuni pagoda (CCO 2008)

Medieval Europe also used a large amount of timber framing for the construction of multi storey timber buildings. The oldest surviving of these dates from the 12th century. The Knochenhaueramtshaus (Figure 1.2) in Hildesheim, Germany was once considered the most beautiful half-timber building in the world, with 8 storeys standing 25.72m tall. A half timber building consists of wood framing filled with plaster, brick or stone. After being almost completely destroyed during the Second World War it was reconstructed in 1987 to the original design.

Figure 1.2: The Knochenhaueramtshaus (Knochenhaueramtshaus.com 2007)

Although timber construction has had a long history throughout the world, in latter years it has been falling behind 'modern' construction material such as concrete and steel.

Modern timber construction largely consists of residential structures. This is mainly due to the use of large wall panels being necessary for seismic resistance. Timber moment connections have previously been avoided due to difficulty of construction and significant costs. However, as global focus shifts towards sustainability and environmental concerns timber construction is an obvious choice for the future.

1.1 Research Objectives and Questions

The objective of this research is to investigate the feasibility of post-tensioned timber multi-storey buildings. This is carried out through the comparison of a case study building designed in concrete, steel and timber. Several research questions will be asked as detailed below:

- How will a timber post-tensioned building be designed?
    1. How will lateral seismic loading be calculated?
    2. What type of flooring will be used?
    3. How will lateral forces be resisted?
    4. What type of connections will be used and how will these connections be designed?
- How will these connections perform under lateral loading?
    1. How will shear at the base of a beam to column or wall to foundation connection be resisted without effecting the rocking motion of the member?
    2. How will floor shear be transfer to the seismic frame?
    3. How will the placement of armour at the beam to column interface influence the moment response of the section?
    4. Is the predicted performance of a beam to column connection using current design procedures accurate?
    5. Is it necessary to place corbels under the seismic beams?
    6. How will the beam to column connection perform with the addition of a floor unit?
- How will a timber post-tensioned structure compare to the current steel and concrete structural design practice?
    1. How will member size compare between the timber and concrete structures?

2. How will construction method and construction time compare between the timber and concrete structures?
3. How will the costs of the timber, concrete and steel buildings compare?

- How will the development of the post-tensioned timber system progress in the construction market?

Is should be noted that this research will not study the seismic performance of case study buildings and will not compare their seismic performance.

1.2 Organisation of Thesis

Chapter 2 covers a literature review of the topic summarising the work performed in timber construction techniques, construction and cost investigations. Chapter 3 outlines the case study building used, presenting an overview of the timber, concrete and steel structures that will be analysed in the report. Chapter 4 shows the loading calculations, and the moment connection and member design for the timber structure. Chapter 5 shows the connection design of the timber building. Chapter 6 offers experimental testing of a selection of key connection details. Chapter 7 presents the construction methods and construction time for both the concrete and timber structures. A detailed cost analysis of the case study structures is in Chapter 8. Chapter 9 presents a business case study for the timber post-tensioned system, proposing the way forward to the method of construction in the multi-storey building market. Chapter 10 presents the conclusions of the thesis.

2 Literature Review

2.1 Timber Construction

Study into the performance of multi storey timber structures can be separated into two categories; light timber framing, and heavy timber construction. The use of light timber framing for housing in New Zealand has been well documented (Garret 1990). This research culminates in NSZ 3604 for the design of light timber framed buildings, which covers non-specific design of buildings fitting within the scope of the standard.

Considerable work has also been performed regarding the design of multi-storey ply shear walls (Stewart 1987, Deam 1997) and hysteretic loops and analytical models have been developed. However, it is required that large walls be used for this method to ensure adequate lateral resistance. This can mean that for medium rise buildings a considerable number of internal walls will be required to resist lateral loading. This in effect 'locks' the internal space of the building making a change of usage impossible. In addition, modern commercial structures often require open plan in internal spaces, making the use of walls impossible.

This method of construction under inelastic lateral loading displays a large amount of pinching behaviour (a significant loss in stiffness due to the inelastic damage around each nail allowing movement), leading to a considerable loss of stiffness during cyclic loading.

The use of cross laminated (cross-lam) panels has also become popular for use in medium rise buildings in Europe (Ceccotti 2008), with rapid erection being realised using pre-fabricated tilt up panels. However, this system still requires an extensive number of walls making it unsuitable for open plan structures.

The development of a multi storey building system for timber relies on the development of either a moment connection or a braced system. Although considerable development in the construction of moment resistant knee joints for portal framed structures has been achieved (Hunt and Bryant 1988, Van Houtte 2003) a feasible frame connection remains illusive. Fairweather (1992) and Buchanan and Fairweather (1993) attempted to develop moment connections concentrating plastic deformation at the interface between the beam and column member. These connections suffered possible brittle failure due to the variability of the Glulam members.

2.2 The Hybrid Connection in Reinforced Concrete

Beginning in late 1985 a research project known as the U.S. PRESSS (Precast Seismic Structural Systems) program at the University of California, San Diego, initiated an extensive amount of research on precast concrete with jointed ductile moment connections. This research studied the combination of mild steel and/or fully or partial bonded post-tensioning (Priestley 1991, 1996; Priestley et al., 1999).

Perhaps the most desirable connection to arise from this research is the ‘hybrid’ connection. This combines the use of unbonded post tensioning and sacrificial mild steel reinforcing. The post tensioning will remain elastic providing a clamping and recentering force while the mild steel yields during cyclic motion provided hysteretic (the amount of energy being released during movement) damping. The behaviour of this connection is characterised buy the ‘flag shaped’ response shown in Figure 2.1.

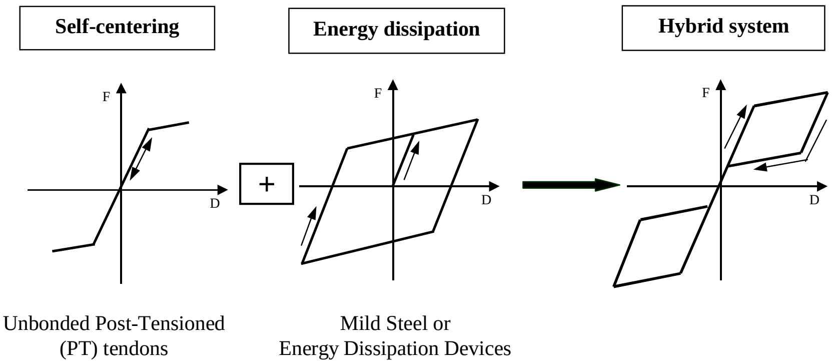


Figure 2.1: Moment rotation response of a hybrid system

(fib 2004, NZS3101:2006 app B)

Performance Based Seismic Engineering (PBSE) is becoming increasingly important in seismic engineering. This combines the consideration of the cost of damage and downtime with the life safety considerations allowing a life cycle cost to be considered rather than simply initial construction costing (Krawinkler 1999). This concept lies at the heart of the hybrid connection construction technique. A hybrid connection concentrates the structural damage after a major event to specific discrete locations, making repair efficient and effective with simple replacement of the damaged yielding elements. Further to this Pampanin et al. (2002) showed that residual displacements post event can have considerable financial consequences, with the possibility of a building being rendered unusable although collapse may not have occurred. The elimination of residual displacement is a further advantage of the hybrid connection. Although the hybrid connection was originally developed for precast concrete Christopoulos et al. (2002) proposed a similar connection for steel connections showing the system to be material independent.

2.3 The development of a Timber Hybrid Connection

In 2004 an extensive research program was launched at the University of Canterbury adapting the precast concrete ductile connection technology for use with Laminated Veneer Lumber (LVL). Shown in Figure 2.2a, LVL is an engineered wood product produced by reducing the raw log into 3mm thick veneers and gluing these veneers together under pressure in the same manner used to form plywood sections, however, the grain is laid parallel. The process in which LVL is produce has the effect of spreading out any defects in the timber (Figure 2.2b) reducing the effects of local weaknesses on the characteristic performance of the member.

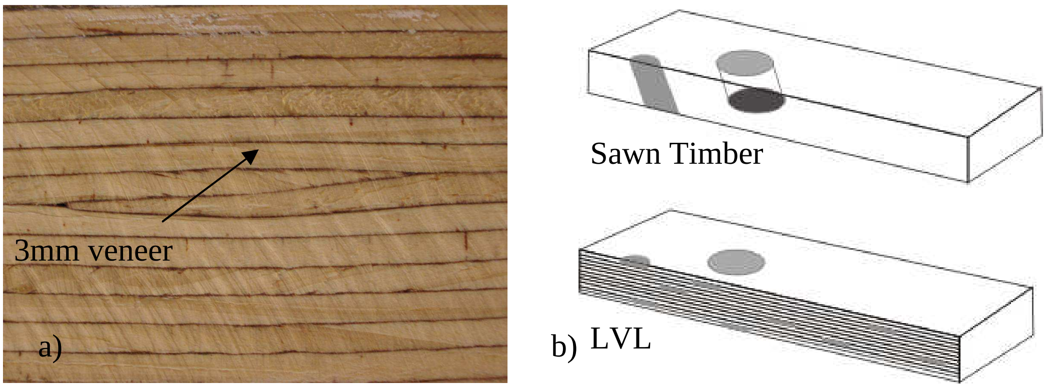


Figure 2.2 a) Laminated Veneer Lumber b) Spreading of defects in LVL

Several subassembly tests have been carried out under both quasi static and pseudo dynamic loading with and without the attachment of dissipative devises. Beam to column (Newcombe 2005, Smith 2006a), wall to foundation (Palermo et al. 2005, Smith 2006b, Smith et al. 2007) and column to foundation (Pasticier 2006, Iqbal 2008) sub assemblies have been tested all with excellent results. This testing has proved that the combination of the hybrid joint and the use of LVL provides an excellent moment connection for framed timber structures as shown in Figure 2.3.

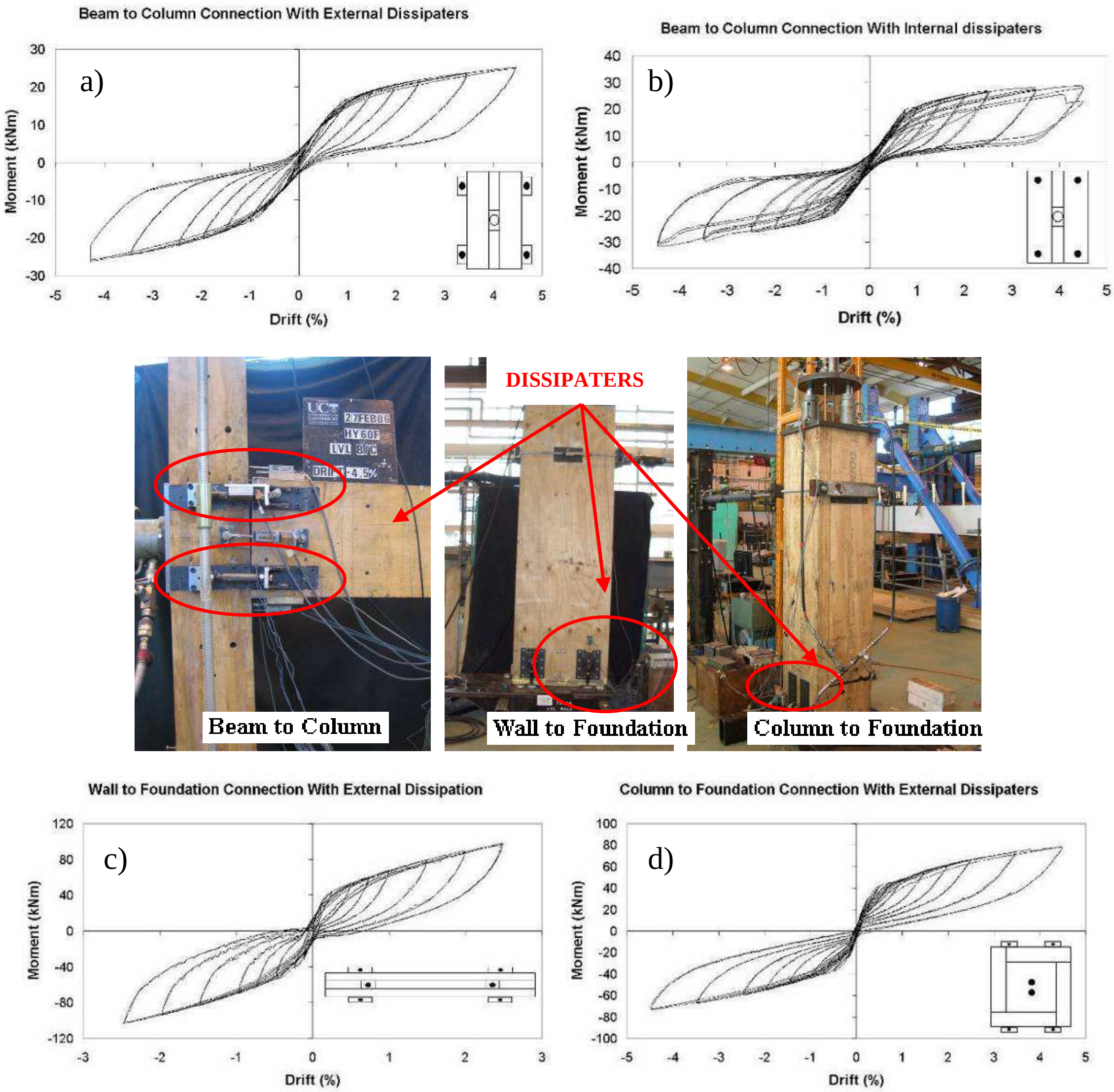


Figure 2.3: Results from moment connection testing: beam to column connection with a) internal dissipation b) external dissipation c) wall to foundation connection d) column to foundation connection (Smith 2008)

The existing design procedure for a jointed ductile precast concrete connection (NZS3101:2006 Appendix B) was modified by Newcombe (2008) making it applicable to the jointed timber connection. In the same contribution the use of Direct Displacement Based Design (DDBD) (Priestley et al. 2007) for post-tensioned timber connections was discussed and a design procedure was proposed. An extended summary of previous testing was also presented in this thesis.

2.4 Costing and Construction of Timber Structures

Tonks (1974) presented an architectural Ph.D thesis describing the feasibility of medium rise office building with glulam frames and plywood sheathed shear walls. The construction and cost of three storey motels has also been discussed and favourable comparisons to similar concrete and masonry structures have been shown (Tonks 1989). Several investigations into the feasibility of multi storey timber buildings were carried out early in the 1990's. Thomas (1991) discussed the design of multi-storey light timber framed buildings, the cost savings made possible though rapidity of construction are also discussed. The re-design of a six storey concrete building using plywood sheathed walls was also presented concluding that under half the construction time is required when compared to the original concrete building. Halliday (1991) discussed the design of 4 to 6 storey timber office buildings using both sheathed walls and large glulam members. The cost and construction was compared to a concrete building, the costs of these two structures were found to be comparable. Due to a substantial amount of prefabrication added to the lightness of timber the total construction time was found to be considerably less.

Although favourable cost and construction time comparisons have been drawn, Fairweather (1992) suggested that the reluctance to use timber as a material for the construction of multi-storey buildings is possibly due to: limited design information for connections, low stiffness resulting in large deflections and fire concerns. For these reasons timber is often perceived as uncompetitive with alternative construction materials. Langenbach (2008) notes that a shortage of steel during World War Two lead to the construction of the largest timber buildings ever made. Therefore, with steel and energy prices rising, it is likely that this same move to timber as a construction material will occur.

3 Case study building

3.1 Actual Building

The case study building used for the project is a six storey structure that is to be built at the University of Canterbury for the Biological Sciences department (Figure 3.1). The actual building is to be constructed in pre-cast concrete.

Figure 3.1: Original concrete building (courtesy of Courtney Architects)

The building has two distinct lateral resisting systems in order to resist loading in both the north-south and east-west direction. In the long (east-west) direction a moment resisting frame will be used. In the short (north-south) direction structural walls will be used.

The structure has been designed to be in the Christchurch region in what can be considered to be a moderate seismic zone. The foundations are in reasonably good conditions considered to be a shallow soil. For all design the current New Zealand design codes have been used. Where these have not been adequate, particularly in the case of the timber structure, other relevant international codes have been utilised.

3.2 Changes to Actual Building

For the purposes of a project run parallel to this thesis (John 2008) some architectural changes have been made to this structure (Perez 2008). It was decided that the basement level of the structure will be removed for the new design and therefore the foundation level was altered by the author to accommodate this change. Although the overall structure has been maintained in the three separate structural designs, some changes were necessary. These are detail in the following paragraphs. A floor plan of this new building is show in Figure 3.2.

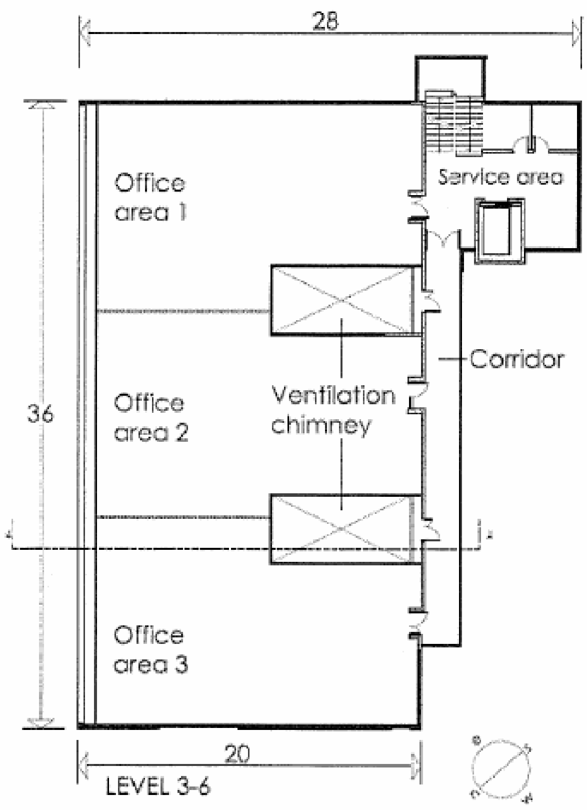
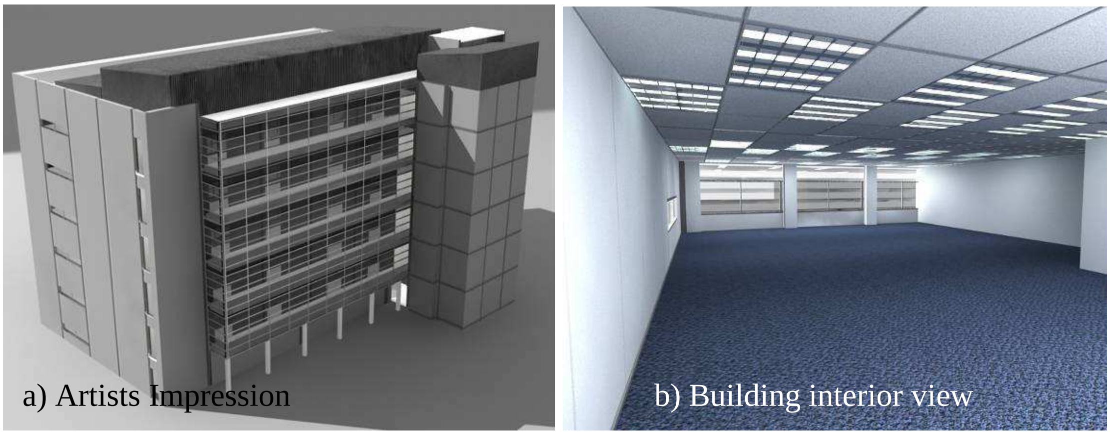


Figure 3.2: Architectural floor plan of case study building (Perez 2008)

3.2.1 New Concrete Structure

Overall the original concrete design will be used with a few minor changes (Figure 3.3), resisting lateral and vertical loading through the use of pre-cast concrete frames and walls. Three precast seismic frames are used. The use of hollow core units spanning in the north south direction will remain.

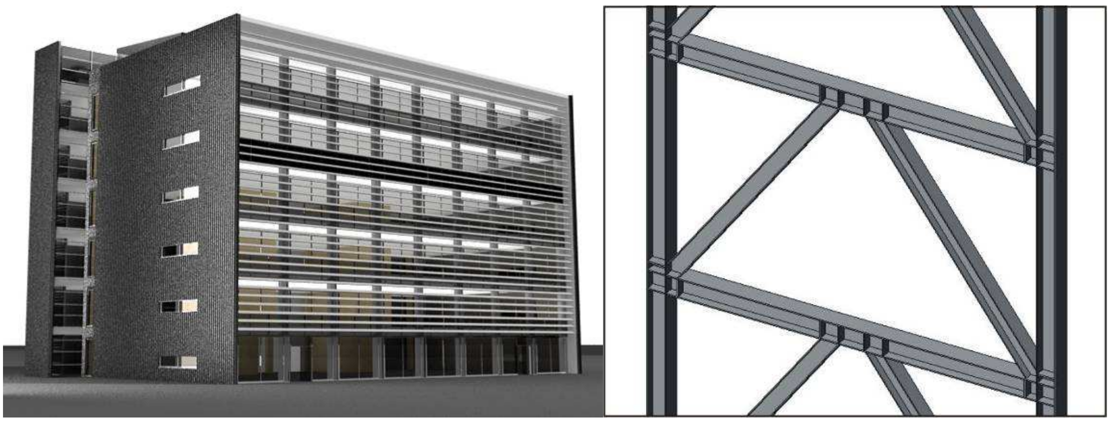


a) Artists Impression b) Building interior view

Figure 3.3: Concrete Building

3.2.2 Steel Structure

The steel structure (Figure 3.4a) had the most significant change in its structural system of the three buildings . The frames and walls are removed and replaced with Eccentrically Braced Frames (EBF's) in both directions (Figure 3.4b). Four of these frames are used in the long and two in the short direction. The remaining members are designed to be only vertically loaded. The flooring will be a Comfloor steel concrete system which places 150mm or reinforced concrete on a 0.9mm corrugated steel decking. The original structural design of this building was performed by Steel Construction New Zealand (SCNZ) (APPENDIX A) and later Holmes Consulting Group was employed to alter and check the lateral resistance design. The author performed the gravity design of the structure.



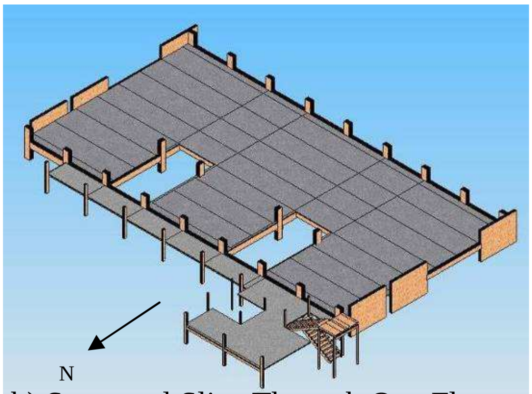
a) Artists Impression b) Eccentrically Braced Frame (EBF)

Figure 3.4: Steel building

3.2.3 Timber Structure

The basic form of the Timber building (Figure 3.5a) will remain similar to that of the concrete structure with the use of frames and walls. The structural system will be altered to use a new method of connection currently under investigation at the University of Canterbury. This combines the use of un-bonded post tensioning cables and sacrificial mild steel in order to achieve force resistance. This system is essentially damage free after a major event and will return to zero residual displacement; these are major advantages for any structural system. The floor units are timber-concrete composite with 65mm of reinforced concrete poured onto 17mm ply sheets which are supported by LVL joists. Figure 3.5b shows a typical flooring plan for the timber structure.

a) Artists Impression



b) Structural Slice Through One Floor

Figure 3.5: Timber Building

4 Structural Design

4.1 Loading Calculations

The following section will describe the loading calculations for the six-storey case study building for all materials. Gravity loadings will first be presented followed by the lateral loadings. Finally the building internal moments and reactions will be displayed for the timber structure.

4.1.1 Timber Building Gravity Loadings

The architectural floor plan for the case study building is displayed in Figure 3.2 above. This shows a total floor plan of 36m x 20m. The corridor and lift shaft area will add additional mass contributing to the total gravity loading in each floor. The building will be classified as an 'office for general use' type structure in accordance with AS/NZS1170.1, therefore a 3kPa live load will be applied. The dead loading from the flooring units is assumed to be 3kPa and a superimposed dead load of 1.0kPa is also added

Using the above floor loads the factored gravity loadings for the flooring can be calculated:

$$\begin{aligned} f_{floor} &= 1.2G + 1.5Q_b \\ &= 1.2(3+1) + 1.5(3) \\ &= 8.7kPa \end{aligned}$$

This loading is used to calculate the demand on a flooring unit, the design of this is shown in Section 4.2.1.1. Tributary areas are used to calculate the proportion of this loading that will be transmitted via axial loading in the seismic and gravity columns.

4.1.2 Building Lateral Loadings

4.1.2.1 Material Independent Direct Displacement Based Design

The Direct Displacement Based Design (DDBD) (Priestley et al. 2007) method has been proposed for the calculation of lateral forces arising from earthquake ground motion. The fundamentals of this design procedure are shown in Figure 4.1.

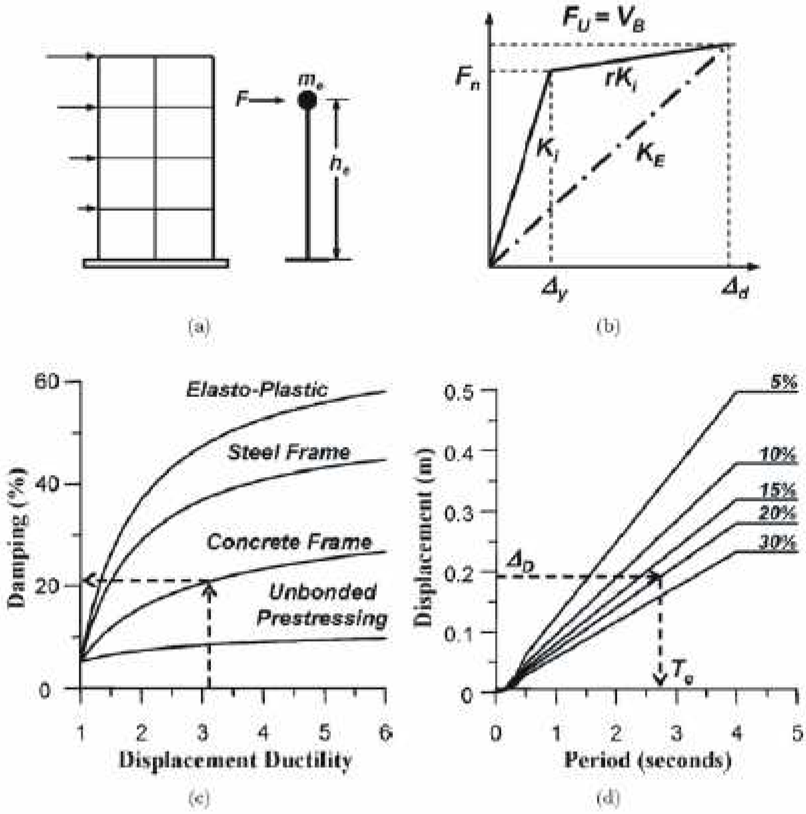


Figure 4.1: Fundamentals of Direct Displacement Based Design Procedure (Priestley et al. 2007)

Shown in Figure 4.1a above the first step of the design process is to convert the multi degree system of the building in to a single degree of freedom system with an equivalent height and mass. As shown in Figure 4.1b this method uses the secant stiffness of the structure in order to calculate the base shear in the building. To do this a design displacement is chosen, this can be based on code requirements or damage considerations. The normalised inelastic mode shape will be selected and should represent the expected predominant displaced shape of the structure. Once the displaced shape and the design displacements have been found the equivalent mass and height of the single degree of freedom system is calculated. The yield displacement must be computed in order to find the likely ductility of the structure;

this will depend on the geometry of the type of structure in question. A damping ratio and displacement reduction factor is then calculated depending of the type of structure used and based on ductility (Figure 4.1c). This damping ratio is used to modify the displacement spectra to account for hysteretic energy release. The reduced displacement spectra (Figure 4.1d) is then used to calculate the equivalent period of the structure and using this the equivalent stiffness and subsequently the base shear is calculated. The equations used for this method are detailed below, as set out in Priestley et al. (2007)

Firstly the normalised inelastic mode shape is calculated:

$$\delta_i = \frac{H_i}{H_n} \qquad \text{For } n \le 4 \qquad (1)$$

$$\delta_i = \frac{4}{3}\left(\frac{H_i}{H_n}\right)\left(1 - \frac{H_i}{4H_n}\right) \qquad \text{For } n > 4 \qquad (2)$$

Where:

δ_i = Dimensionless normalised mode shape at floor i

H_i = Height at floor i (m)

H_n = Total height of the structure (m)

n = Total number of storeys in the building

This mode shape is then related to the displacement at the critical storey:

$$\Delta_i = \delta_i \left(\frac{\Delta_c}{\delta_c}\right) \qquad (3)$$

Where:

Δ_i = Displacement at floor i (m)

Δ_c = Displacement at critical storey (based on the max inter-storey drift) (m)

δ_c = Value of dimensionless mode shape at critical storey

The design displacement of the single degree of freedom structure is then calculated:

$$\Delta_d = \frac{\sum_{i=1}^{n}\left(m_i \Delta_i^2\right)}{\sum_{i=1}^{n}\left(m_i \Delta_i\right)} \qquad (4)$$

Where:

Δ_d = Design displacement of single degree of freedom structure (m)

m_i = Mass at storey i (tonnes)

Next the effective height and effective mass of the single degree of freedom structure found:

$$H_e = \frac{\sum_{i=1}^{n}\left(m_i \Delta_i H_i\right)}{\sum_{i-1}^{n}\left(m_i \Delta_i\right)} \qquad (5)$$

$$m_e = \frac{\sum\left(m_i \Delta_i\right)}{\Delta_d} \qquad (6)$$

Where:

H_e = The effective height of the single degree of freedom structure (m)

m_e = The effective mass of the single degree of freedom system (tonnes)

The yield displacement is necessary in order to calculate the ductility of the structure. The increased stiffness of a hybrid connection reduces the yield drift of a hybrid framed structure (Priestley et al. 2007). This value is approximated by:

$$\theta_y = 0.0005 \frac{L_B}{h_b} \qquad (7)$$

Where:

θ_y = Yield rotation

L_b = Beam length (m)

h_b = Beam height (m)

The system ductility is then calculated:

$$\mu = \frac{\Delta_d}{\Delta_y} \tag{8}$$

Using this ductility the reduction factor is calculated. For hybrid structures the hysteretic damping is provided by the addition of some form of sacrificial device. The displacement reduction factor (η_ξ) is calculated as follows:

$$\eta_\xi = \sqrt{\frac{7}{2+\xi_{eq}}}\,\% \tag{9}$$

Where:

ξ_{eq} = Equivalent viscous damping ratio

The effective period (T_e) is found by entering the selected displacement spectrum with the value corresponding to a given calculated viscous damping.

The equivalent stiffness (K_e) of the structure is then found:

$$K_e = \frac{4\pi^2 m_e}{T_e} \tag{10}$$

From this the base shear can be computed:

$$F = V_{Base} = K_e \Delta_d \tag{11}$$

This base shear is then distributed up the building in proportion to both the mass and displacement of each floor:

$$F_i = V_{Base} \frac{(m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)} \tag{12}$$

4.1.2.2 DDBD for Post-Tensioned Timber Frames

The use of this design procedure is deemed to be direct due to the estimation of the yield rotation meaning that iterations in the load calculations are not necessary as an adequate structural ductility is estimated. This estimation is simply based on geometric and material properties and provides good estimates for design purposes. This estimation is critical to the calculation of reduction in base shear arising from hysteretic damping. Unfortunately the yield rotation of post-tensioned timber frames depends strongly on the seismic forces applied (Newcombe 2008). And therefore the process is no longer considered direct due the iterations that must be performed, therefore a modified procedure is proposed (Figure 4.2). This is due to the joint deformation of the timber connection contributing significantly to the yield rotation. In this same contribution it has been shown that this contribution can be up to 50% of the elastic deformation due to the low shear modulus of LVL. It is important to note that these calculations are based on a conservative assumption and experimental validation has not been performed. An expansive study of the effect of the elastic deformation can be found in Newcombe (2008), however, it has been neglected for the remainder of this case study.

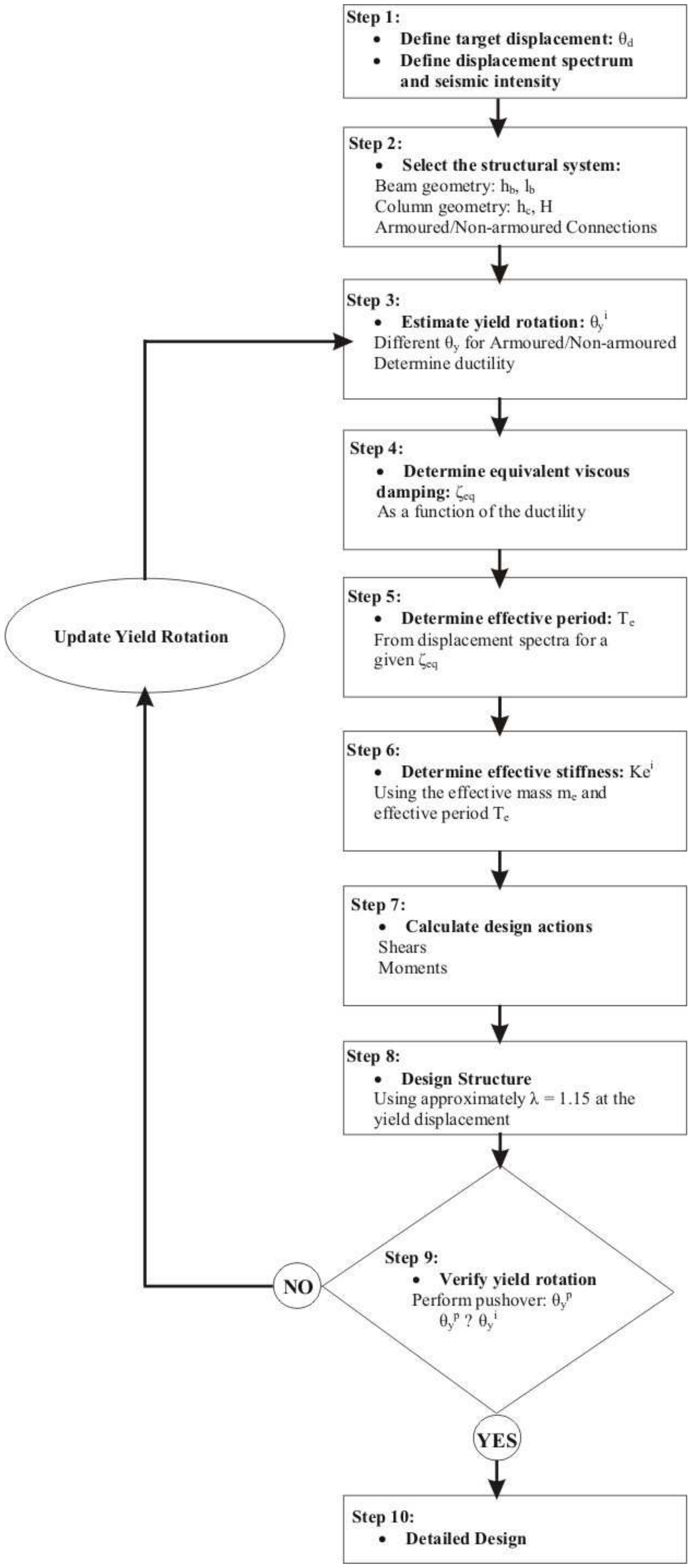


Figure 4.2: DDBD procedure for post-tensioned timber connection (Newcombe 2008)

4.1.3 Lateral Load Calculations for Case Study Timber Building

The DDBD procedure is used for the lateral load calculations for the six storey case study building. For this the seismic mass at each floor must be calculated:

$$F = G + Q_e$$
$$= G + \Psi_c Q$$
$$= G + \Psi_c \Psi_a Q_b$$

Where:

$$\Psi_a = 0.3 + \frac{3}{\sqrt{A}}$$

Ψ_c = Combination factor for imposed 'live' loads

Ψ_a = Area reduction factor

Using these values the imposed loads show in Table 4.1 were calculated.

Table 4.1: Floor loadings for case study building

Level	Area (m^2)	Ψ_a	Ψ_c	G	Q_b	W_i (kN)
Roof	663	0.42	0	3.5	3	2321
5	726	0.41	0.4	3.5	3	2899
4	726	0.41	0.4	3.5	3	2899
3	726	0.41	0.4	3.5	3	2899
2	744	0.41	0.4	3.5	3	2970
1	766	0.41	0.4	3.5	3	3056

The above values in Table 4.1 are altered slightly from that of the original estimates of floor seismic weight used in the design. The values in Table 4.1 were calculated as the final architectural layout was finalised. In addition to this the conclusions of Newcombe 2008 have lead to an increased knowledge of the application of the DDBD procedure and specifically the effect of equivalent viscous damping.

Due to these improvements the base shear values used in the original design were reassessed. It was found that the additional information increased the total base shear

12% from an initial value of 1600kN to 1800kN. This increase is deemed to not be significant and therefore the original values, and hence design, is still acceptable.

4.1.3.1 Calculation of Internal Design Actions

Once the total base shear is found it is distributed up the building following the assumed first mode displacement and weighted by the floor mass in accordance with Equation 11 in Section 4.1.2.1.

In order to calculate the given actions a simplified equilibrium based approach was used (Priestley 2007). DDBD allows the designer to select the amount of base shear to be distributed into each column, however, it was decided that each column will be subjected to an equal moment. The base moments are then applied in accordance with the following equation:

$$M_{Ci} = 0.6V_{Ci}H_{1i}$$

Where:

M_{Ci} = Moment at base of column i

V_{Ci} = Base shear taken by column i

H_{1i} = 1st storey height of column i

Using this method it is possible to allow for the reduced stiffness of the tension column as this reduced stiffness will mean that less seismic forces are attracted. This has not been considered for the design of the case study building. The equilibrium approach is used to calculate the critical moments at each level. These moments are displayed in Table 4.2 with the Bending Moment Diagram (BMD) shown in Figure 4.3.

Table 4.2: Column face design moments for timber frame

Floor	Height (m)	Moment (kNm)
Roof	22.86	56.0
5	19.05	96.8
4	15.24	131
3	11.43	158
2	7.62	177
1	3.81	188

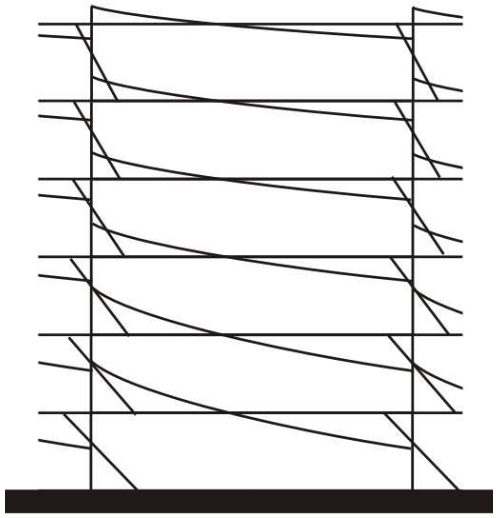


Figure 4.3: BMD for timber building

The overturning moment at the base of each column is 275kNm.

4.1.3.2 Moment Calculation in Wall Direction

For the wall lateral loading calculations the same displacement based design method was used, however the displaced shape of the structure was altered to represent the flexural deformation of the wall system. The total overturning moment for the building is divided equally amongst the three structure walls with 8242 kNm overturning moment at the base of each wall.

4.1.4 Force Calculations for Concrete and Steel Alternative Designs

The lateral loading calculations for the concrete and steel buildings were performed in accordance with a traditional force based approach which is considered to represent current practice techniques. The force calculations Concrete and Steel building were performed buy Lovell Smith & Cusiel Consulting and Steel Construction New Zealand respectively. The equivalent static method was used to distribute the forces up the height of the building and an elastic design package was used to calculate the internal forces.

4.2 Frame and Wall Design for Timber Building

Once the internal vertical and horizontal forces are calculated the member load paths, member sizes and moment connections are found. First the gravity load paths are established through the use of prefabricated timber-concrete composite flooring panels. The loadings from these are collected into large gravity beams which are then connected to columns. For the lateral resistance a new form of timber moment connection is used. The connection details are outlined in Chapter 5.

4.2.1 Gravity Loadings

4.2.1.1 Timber-Concrete Composite Floor Panels

Presently at the University of Canterbury a new form of timber-concrete composite flooring is being developed. This consists of prefabricated timber panels fabricated off-site with 65 mm concrete topping cast on site. The timber panels are made from two adjacent 63×400 mm LVL joists spaced at 1200 mm centres with a nailed plywood sheet (Figure 4.4). Notches cut from the joists will be filled by concrete, reinforced by one coach screw at the centre of each notch, to give composite behaviour. 10mm steel mesh at 200 centres is placed inside the concrete to control shrinkage cracking. This composite behaviour allows a significant increase in stiffness of the system. The concrete topping also improves the acoustic separation between intertenancy floors. For further information regarding this system, and the design of it, refer to Buchanan et al. (2008) and Yeoh et al. (2008).

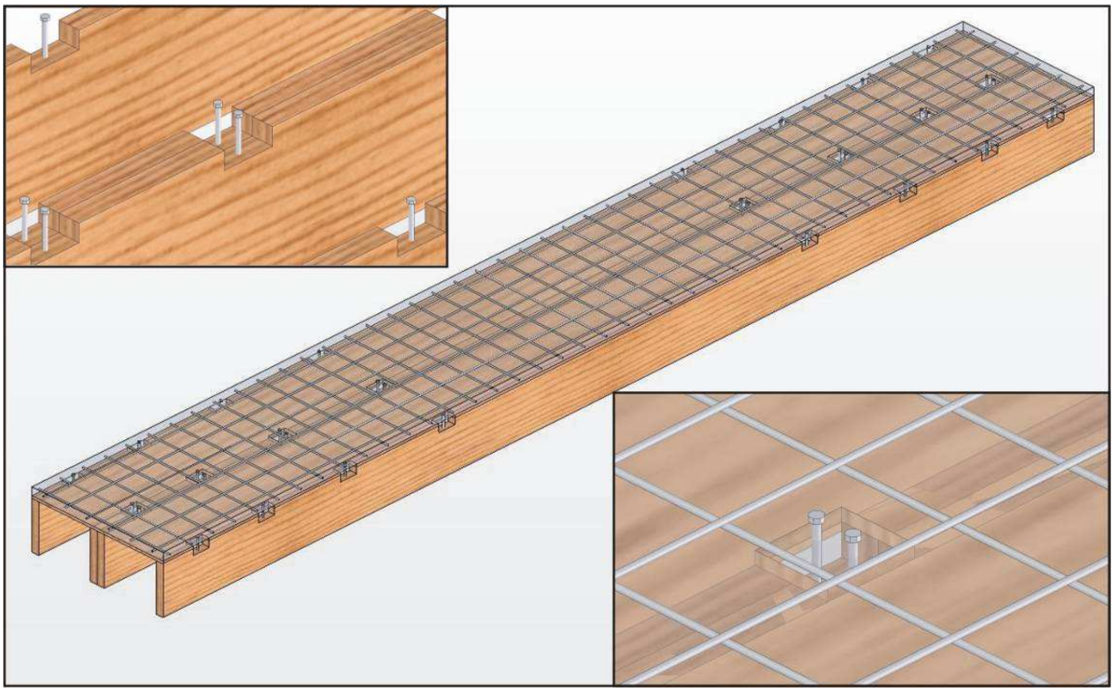


Figure 4.4: Timber-concrete composite flooring unit

4.2.1.2 Gravity Beam and Column Design

The flooring is connected to four large LVL gravity beams in the centre of the structure. In order to enable the reduction of the size of these beams these are tied into the floor slab through the use of notched coach screws placed in the top of the beam member. This allows composite action to form and reduces deflection by increasing the stiffness of the member.

Four gravity columns are positioned near the centre of the building. These collect the forces from the gravity beams and have been designed in accordance with NZS 3603:1993 for an axially loaded timber member.

4.2.2 Seismic Loadings

4.2.2.1 Development of the Concrete Hybrid Connection

As described in Section 2.2 recent developments in the field of seismic design have lead to the development of damage control design philosophies and innovative seismic resistant systems. In particular, jointed ductile connections for precast concrete structures (Priestley 1991, 1996; Priestley et al., 1999; Pampanin, 2005) have been implemented and validated. These solutions rely on a discrete dissipative mechanism placed in specific locations in the structure.

A precast concrete seismic resisting system developed in the U.S.-PRESSS program (PREcast Seismic Structural System), coordinated by the University of California, San Diego, for frame and wall systems has been shown to be particularly effective. This system, referred to as the hybrid system, combines the use of unbonded post-tensioned tendons with grouted longitudinal mild steel bars or any form of dissipation device. While the post-tensioning provides a desirable recentering characteristic, the dissipation devices allow adequate energy absorption by the system. During lateral movement a controlled rocking will occur at the beam to column, wall to foundation or column to foundation interface.

4.2.2.2 Design of a Concrete Hybrid Moment Connection

A detailed design procedure for the moment calculation of a hybrid joint has been devised (Pampanin et al. 2001) and is presented in Appendix B of the New Zealand concrete code (NZS 3101:2006). It has been suggested that this procedure can be simply applied to the design of a timber hybrid connection (Priestley 2007) provided a few simple considerations are made (Newcombe 2008).

This design procedure is outlined below and summarised below in Figure 4.5:

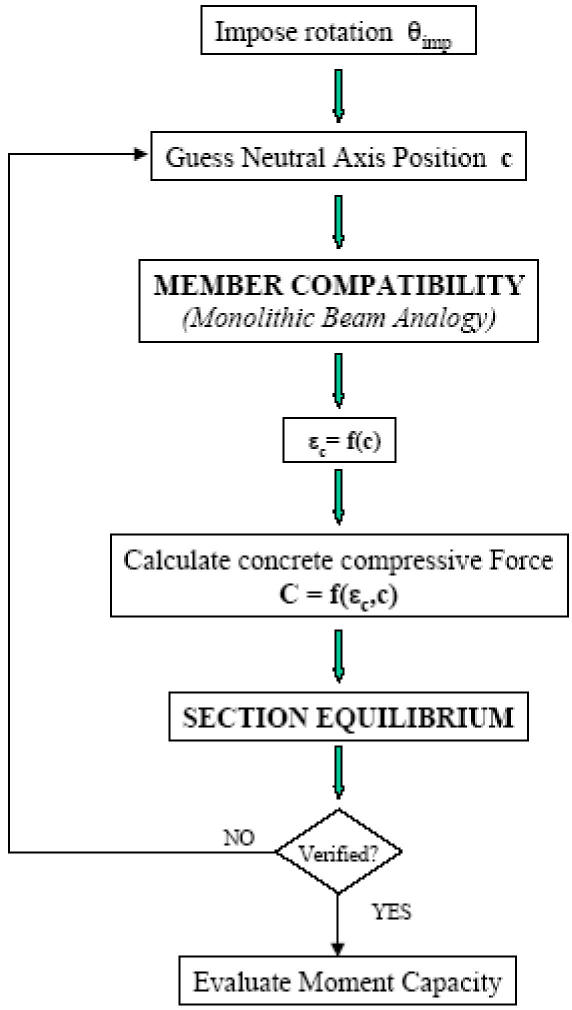


Figure 4.5: Moment rotation procedure for jointed ductile connections (Pampanin et al. 2001)

Select a rotation θ_{imp}

For a beam to column connection an effective rotation (θ_{eff}) is used:

$$\theta_{eff} = \theta_{imp}\left(1 - \frac{d_c}{L_{CL}}\right)$$

Where:

d_c = Column depth

L_{CL} = Length between column centrelines

Guess neutral axis depth c

Section compatibility

Using the imposed effective rotation and guessing the neutral axis depth the strains in both the post tensioning and energy dissipation can be calculated (Figure 4.6)

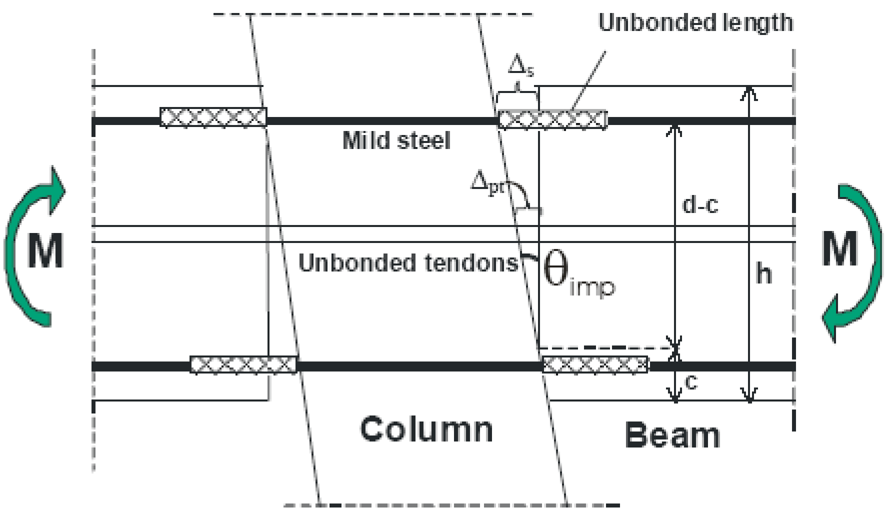


Figure 4.6: Internal beam to column joint mechanism
(Pampanin et al. 2001)

Using the approximated strains (ε_{pt}) induced the forces in the post tensioning (T_{PT}) are calculated:

$$\varepsilon_{pt} = \frac{\Delta_{pt}}{l_{ub}}$$

$$T_{PT} = \varepsilon_{PT} E_{PT} + T_{PT,i}$$

Where:

Δ_{pt} = Change in displacement of post tensioning

l_{ub} = Unbonded length of post tensioning

E_{PT} = E Modulus of post tensioning

$T_{PT,i}$ = Initial post tensioning force

The strains in the energy dissipaters are also calculated:

$$\varepsilon_s = \frac{(\Delta_s + 2\Delta_{sp})}{l'_{ub}}$$

Where:

Δ_s = Change in displacement of dissipater steel

Δ_{sp} = Change in displacement of dissipater due to strain penetration

l'_{ub} = Unbonded length of dissipater

For the concrete system the strain penetration value (increased strain beyond inside the anchorage length of the bar) can be taken as 0.022 $f_y d_{bl}$ (*mm*), a tentative suggestion for this value in a timber connection is presented in Section 4.2.2.3. On calculation of the steel strain the force in the dissipation is calculated using the chosen stress strain relationship.

The force in the concrete compression area is calculated using the Monolithic Beam Analogy (MBA) (Pampanin et al. 2001). This analogy uses a comparison between the jointed ductile hybrid member and an equivalent strain compatible monolithic member in order to find the strain in the concrete. Originally this was only evaluated for response in the plastic domain, however, Palermo (2004) proposed the modified monolithic beam analogy (MMBA) extending it to capture the post - decompression and pre-yielding behaviour.

Section force compatibility is applied and neutral axis (c) is updated

Using equilibrium the force is checked

$$C - T_S + C_S = T_{PT}$$

Iteration is performed varying c until equilibrium is satisfied

The moment capacity is computed

Once the exact forces are found the moment capacity is calculated for the imposed rotation.

On the calculation of the moment capacity it is necessary to ensure that the recentering of the connection will occur. For this the moment contribution of the post tensioning steel is simply compared to that of the dissipating element. For this comparison the following equation is used:

$$\lambda = \frac{M_{pt} + M_N}{M_s} \leq \alpha_o$$

Where:

α_o = 1.15 when strain hardening of the dissipater is not considered

4.2.2.3 The Adaptation of Hybrid Connection to Timber

Newcombe et al. (2008) has proposed necessary modifications to this design procedure in order to make it applicable to the post-tensioned timber system. A summary of these considerations is presented in the following paragraphs.

Effects of LVL Material Properties

As timber is a natural material the stress strain relationship can vary markedly depending on the forest stock used and even between cuts inside the tree itself. The process of Laminated Veneer Lumber (LVL) manufacture means that the material properties are less spread causing an increase in the characteristic stiffness (Figure 4.7a). Although this increase occurs LVL is still an anisotropic material with differing stress and strain properties depending on the loading direction (Figure 4.7b).

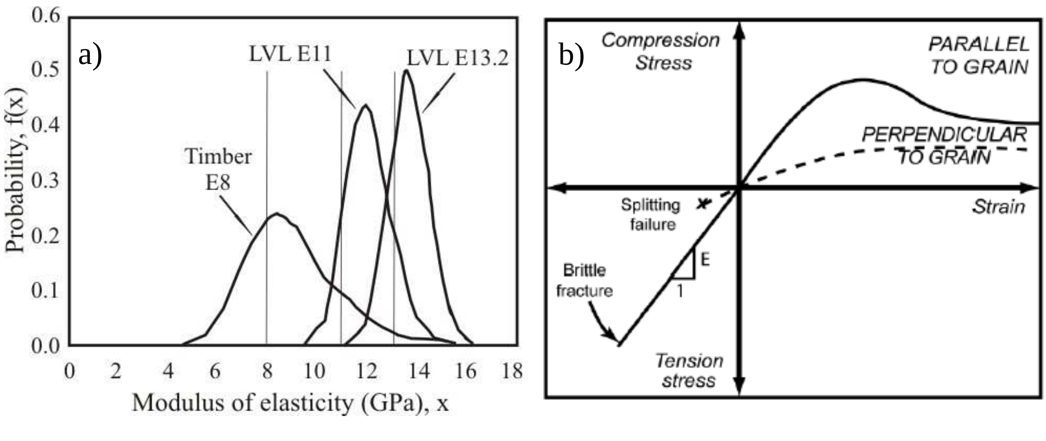


Figure 4.7: a) Modulus properties of LVL compared with standard timber b) Stress strain relationship for timber (Buchanan 2007)

This stress relationship is of specific importance in a beam to column connection as the stress will be applied both perpendicular and parallel to the grain. Observations during compression testing of interacting specimens (Davis 2006) have shown the presence of an intermediate stiffness between that of the perpendicular and parallel to grain stiffness displaying their interaction under compressive loading. From this testing (Figure 4.8) this intermediate stiffness has been observed to equal 1400MPa.

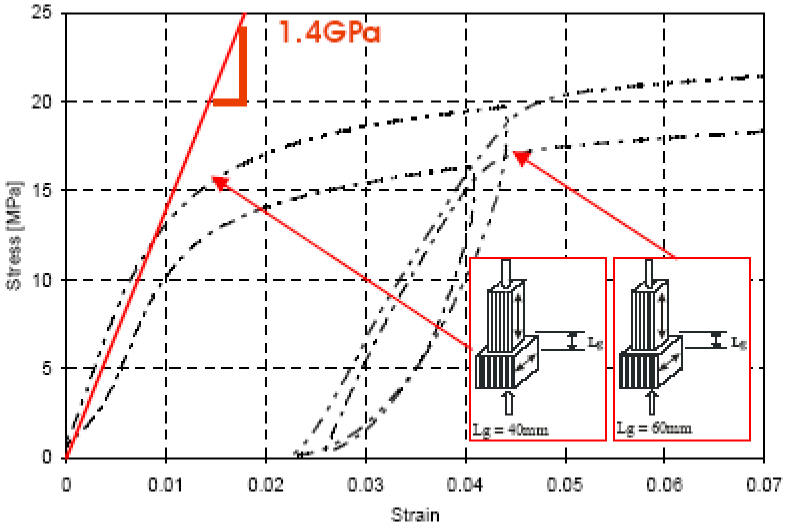


Figure 4.8: Investigation into grain interaction stiffness (Davis 2006)

Further to these considerations it has been observed that significant variations in initial stiffness occur due to an end effect present during compression loading. It is important to note that this is not specific to LVL and occurs in the loading of all timber members. This effect causes a reduction in the elastic modulus, in order to estimate the elastic modulus moment rotation models have been calibrated against sub-assembly tests. Leading to the following equations being tentatively suggested (Newcombe et al. 2008):

For Wall and Column to foundation connections:

$$E_{con} = 0.55E_t$$

For Beam to Column connections:

$$E_{con} = 0.096E_t$$

Where:

E_{con} = The E modulus of the connection

E_t = The mean E modulus of timber

The beam to column equation is for that of an unprotected column, hence the timber modulus perpendicular to the grain is a significant factor. If the face of the column is protected (eliminating the perpendicular to grain effect) the wall to foundation equation is used.

Lastly it has been shown by Newcombe (2008) that the stress strain relationship for LVL can be adequately captured by Popovics concrete stress strain model (Popovics 1973).

Strain penetration, Bond Degradation and Slippage

For the case of internal dissipation the most common system is the use of internally epoxied bars. As the strength and stiffness of this type of connection are reliant on several different parameters (Buchanan and Deng 1996) it is not possible to simplify the behaviour to the same extent as reinforced concrete and modified relationships have been tentatively proposed (Newcombe et al. 2008):

For Beam to Column connections

$$\Delta_{sp} \approx \varepsilon'_s \left(0.0064 f_y d_b\right)$$

For Wall to Foundation connections

$$\Delta_{sp} \approx \varepsilon'_s \left(0.024 f_y d_b\right)$$

Where:

Δ_{sp} = Length of strain penetration

ε'_s = Strain in energy dissipater

d_b = Diameter of bar (outside fuse length)

Note that the wall to foundation equation is only the strain penetration in the timber and the strain penetration contribution from the connection to the foundation must also be considered.

The level of bond degradation expected is difficult to predict and therefore the following design recommendations should be adhered to in order to reduce its effect:

- The embedment length must be greater than 16db. The embedment length used in the experimental testing by Senno et al (2004) was 10db and the Timber Structures Standard (NZS3603:1999) specifies a minimum required embedment length to ensure glue-line failure does not occur of only 5db. However, due to the cyclic demands on the PRESSS-Timber connections greater conservatism should be applied.

In addition, it has been demonstrated by experimental testing that $16d_b$ is sufficient to ensure there is not significant bond degradation and bond slip.

- The minimum edge distances must satisfy recommendations for epoxied connections (Buchanan 2007; Van Houtte 2003); edge distance > $1.5d_b$.
- A low viscosity epoxy should be used to ensure full bond
- The glue-line thickness should be between 1-2mm over the bonded region of the dissipater.
- Laminated Veneer Lumber should be used unless extensive experimental testing is performed on the connections. The epoxied connections are critical components, high variability in the timber strength may result in an unexpected failure mechanism (Fairweather 1993) or large variations deformation due to strain penetration.

(Newcombe 2008)

Modification of the Monolithic Beam Analogy for Timber

The only variation in the MBA for timber is required in the plastic domain of the displacement by altering the equivalent plastic hinge length. However, due to the low stiffness of the material a large proportion of the displacement will occur in the elastic range and therefore the MMBA should be used accounting for this region. Newcombe et al. (2008) derives the equivalent hinge length from strain penetration relationships:

Wall or Column to foundation (neglecting foundation contribution)

$$L_P \approx 0.024 f_y d_b$$

Wall or Column to foundation (with concrete foundation)

$$L_P \approx 0.024 f_y d_b + 0.022 f_y d_b$$

Beam to Column

$$L_P \approx 0.064 f_y d_b$$

4.2.3 Material Properties

4.2.3.1 Laminated Veneer Lumber

As mentioned in Section 4.2.2.3 the process in which LVL is made reduces the distribution of strength and stiffness increasing a characteristic strength of the material. Although this is the case most LVL manufactures produce there own specific material which leads to differing material properties. For the design of the six storey case study building Hyspan produced by Cater Holt Harvey Ltd. will be used. The material properties are shown in Table 4.3:

Table 4.3: Material properties for HySpan LVL

Elastic Moduli		**(MPa)**
Modulus of Elasticity	E	13200
Modulus of Rigidity	G	660
Characteristic Strengths		
Bending	f'_b	48
Tension Parallel to Grain	f'_t	33
Compression Parallel to Grain	f'_c	45
Shear in Beams	f'_s	5.3
Compression Perpendicular to Grain	f'_p	12
Shear at Joint Details	f'_{sj}	5.3
Density		620kg/m^3

Further to this information certain values not found in Futurebuilds' technical manual were required. The perpendicular to grain compression stiffness is 660 MPa (Newcombe 2008) and the tension perpendicular to grain strength is 0.6MPa. (Banks 2007).

4.2.3.2 Dissipaters and Post Tensioning Tendons

Further to the properties of the LVL, Some steel properties were required and are shown in Table 4.4:

Table 4.4: Steel properties

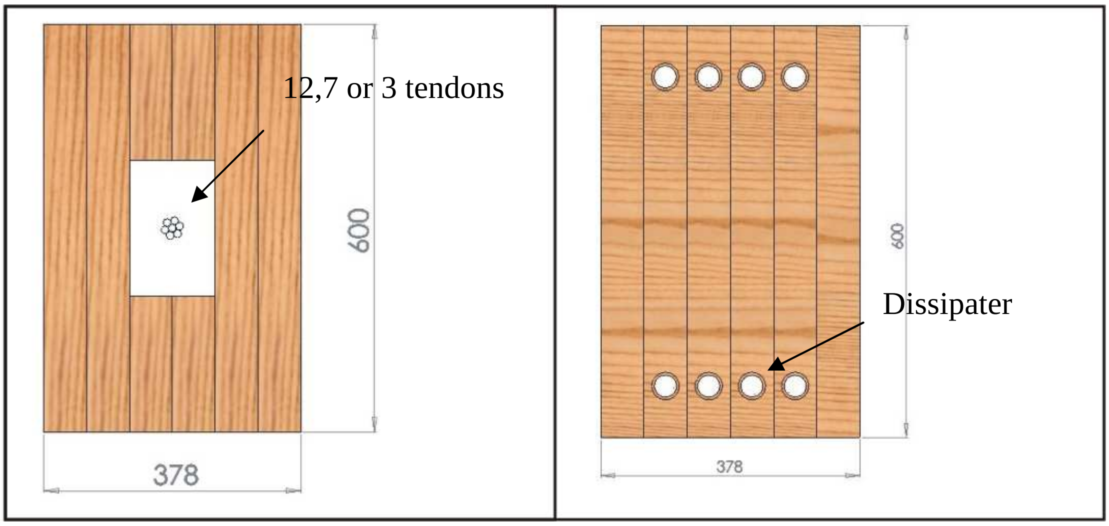
Post Tensioning			
Yield Strength	f_y	1530	MPa
Ultimate Strength	f_u	1860	MPa

Elastic Modulus	E	200	GPa
Mild Steel Dissipaters			
Yield strength	f_y	340	MPa
Yield Strain	ε_y	0.0015	
Elastic modulus	E	200	GPa

4.2.4 Connection Design

The beam to column connection was designed to have a moment capacity greater than 213kNm at the design drift of 1.4%. The finalised design is a 600 x 378 LVL beam with varying post-tensioning up the building. The first two floor levels require twelve 0.5 inch tendons with an initial tensioning of 70% of the yield stress (1057kN). Seven 0.5 inch tendons will be use in the next two levels, and three 0.5 inch tendons used in the last two levels (Figure 4.09a). Steel armouring is placed at the face of each connection to reduce the effect of the low perpendicular to grain stiffness of LVL. As it is likely that the frame will remain nearly elastic with minimal gap opening during the design seismic event, dissipation will not be used in the connection.

The column member (Figure 4.9b) is the same size as that of the beam member. The moment demand at the base of each column is 136kNm. Mild steel energy dissipation is used to provide both hysteretic damping and additional strength. No post tensioning is necessary due to the gravity loading from the structure added to the recentering contributions from the beams being adequate in recentering the building. The energy dissipation has a width of 25mm with an unbonded length of 200mm.



a) Beam member b) Column member

Figure 4.9: Frame sections

The wall base moment is considerably larger than that of the columns with a moment demand of 8242 kNm. 4000mm x 252mm walls will be used. 50mm MacAlloy bars are placed in two ducts (Figure 4.10). 32mm diameter fused internal dissipation is also used with a fuse length of 500mm.

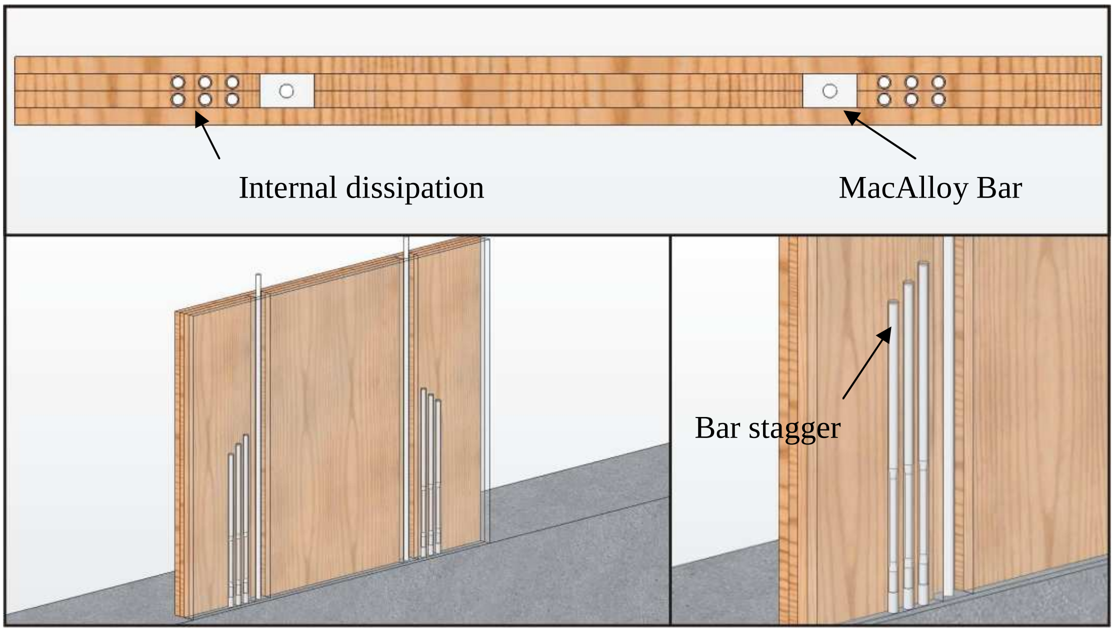


Figure 4.10: Wall section

Table 4.5 shows a comparison between the sizes of the timber beam, column and wall sections with that of the original concrete frame design.

Table 4.5: Member comparison between timber and concrete buildings

	Timber		**Concrete**	
	Dimension (mm)	Area (m^2)	Dimension (mm)	Area (m^2)
Beam	600 x 378	0.23	800 x 400	0.32
Column	600 x 378	0.23	800 x 400	0.32
Wall	4000 x 252	1.01	4300 x 200	0.86

As Table 4.5 shows the timber members are comparable to that of the original concrete design. The wall area for the timber building is 15% larger than that of the concrete wall. However it can be seen that the beam and column areas for the timber structure are 28% less than that of the concrete structure. Therefore, due to the considerable lighter mass in comparison to concrete (6.2 kN/m^3 versus 24 kN/m^3) and the comparable member sizes, significant mass savings occur.

To further this mass saving, as the size of the beam member is governed by the demand moment at the beam to column connection, it is possible to remove a large portion of the central section of the beam (Figure 4.11). It is not suggested that this be done to the column due to increased deflection issues arising from the reduced section. This will also reduce the amount of material required and therefore the cost of the member.

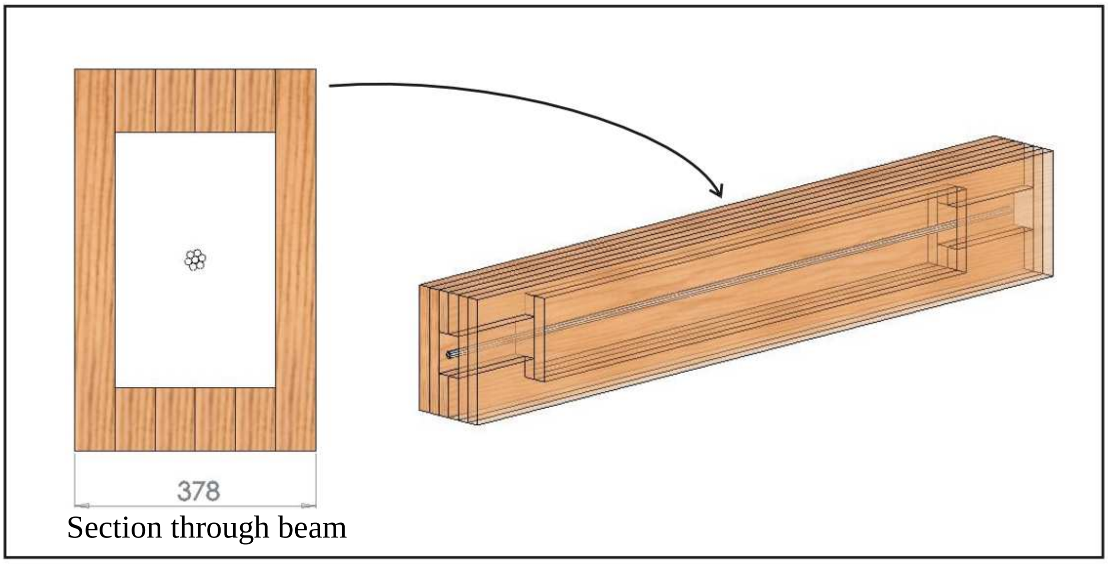


Figure 4.11: Cavity in beam section

4.3 Foundation Design

As mentioned in Section 3.2 the foundation level of the case study building was altered from the original design of the concrete structure. This meant that a re-design of the foundation level was required.

It was assumed for design purposes that the building is situated on a 0.5kPa soil. For the timber building beam foundations are placed under both the seismic frame and walls, with pad foundations under the four central gravity columns (Figure 4.13). This layout was also used for the concrete building however a slight increase in the capacity of the foundations was necessary. For costing analysis, the foundations for the steel building were considered to be the similar to that of the timber building due to the similar masses of the structures. However, upon consultation with Holmes

Consulting Ltd (APPENDIX B) a 15% increase was added due to the considerable uplift forces applied by the eccentrically braced frames.

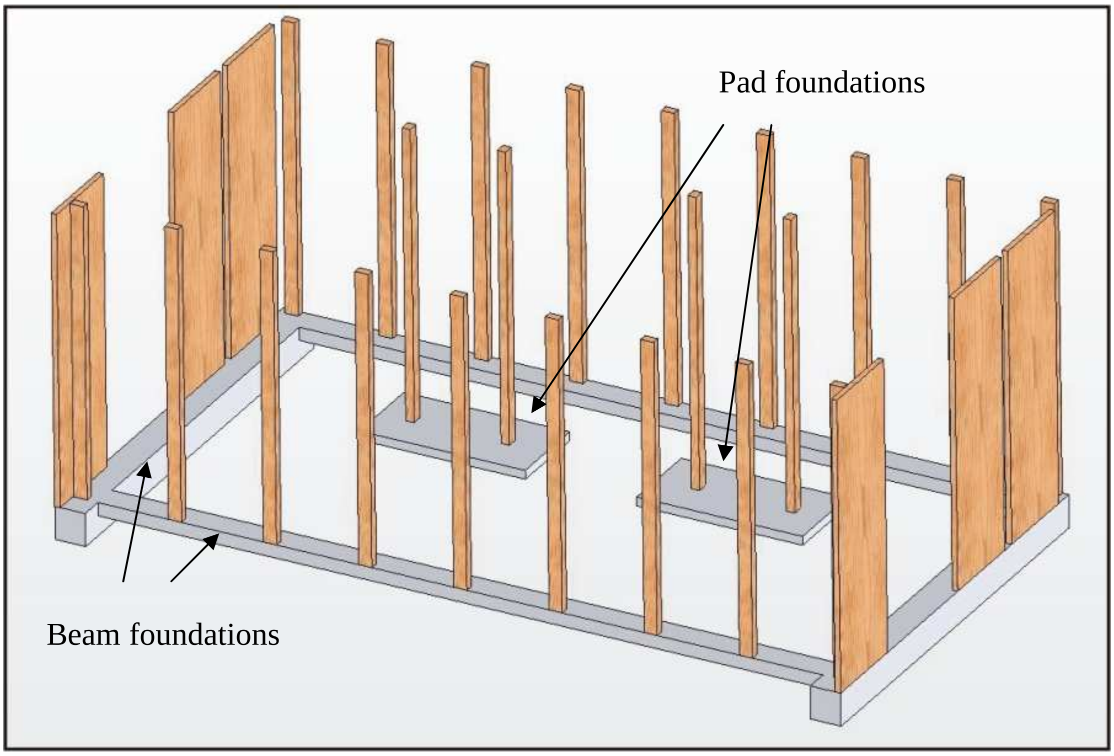


Figure 4.12: Foundation layout for timber building

Calculations of the foundation size for the timber and concrete buildings found that the size of the foundations required are comparable. This is unexpected due to the lightness of the timber building intuitively leading to a reduction in foundation size. However, the foundation size was principally governed by the overturning moment applied by vertical members during a seismic event, and not gravity loading. It can be expected that a significant reduction in foundations between the timber and concrete structures will occur if a building is gravity dominated or is situated in soft soil.

5 Connection Design of Case Study Timber Building

On completion of the seismic design of the six storey biological sciences building attention was turned to the connection detailing of the structure. The combination of the timber-concrete composite flooring system and the innovative post-tensioned timber system enables considerably larger spans to be achieved compared to traditional timber construction. These longer spans cause increased gravity loadings to be placed onto members and therefore connections. Although the increase in the characteristic strength of LVL means that these larger loadings can be accommodated, the connection details are required to reach capacities that have previously not been achieved in timber structures.

Further to the complication arising from the choice of material, the nature of the seismic resisting system adds additional difficulties to the detailing of the structure. As the system undergoes a controlled rocking motion during a significant seismic event it is important that: 1) connections do not hinder this movement and 2) they sustain a minimal amount of damage.

The following chapter describes the solutions devised for the gravity and seismic member connections for the biological sciences building. Although these connections have been design for a specific building it is noted that they can be easily applied to other large span timber structures.

5.1 Joist Hanger Design

The use of joist hangers for timber construction is common practice for both residential and low rise commercial timber construction. The application of joist hangers allows quick construction and the ability for mass production of the product means that the cost of a single element can be kept to a minimum.

A previous study into the use of joist hangers for long span construction has suggested that due to the large amount of nails required that the system is not feasible (Halliday 1991). Although this is true, the development of Type 17 screws has shown that considerably larger characteristic shear strengths can be achieved (Guant 2007). This joist hanger arrangement is shown in Figure 5.1.

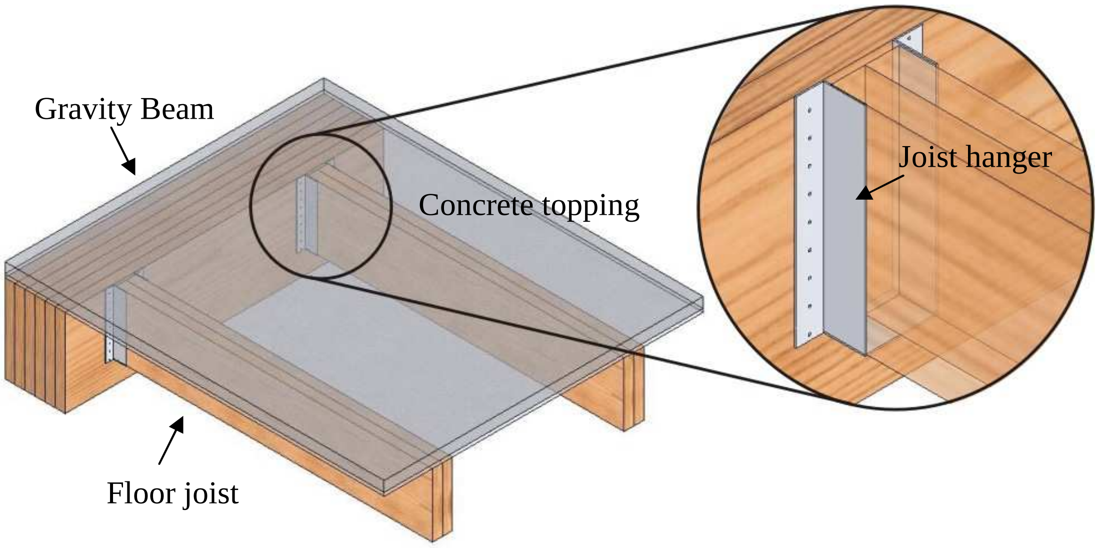


Figure 5.1: Joist hanger connection

The joist from the composite floor bears directly onto the joist hanger with two horizontal nails being applied to the joist for stability during construction. Type 17 (Grade 300MPa Steel, 5.15mm shank diameter) screws of 140mm length are required to take the maximum shear loading in accordance with NZS 3603. A gap is left between the face of the joist and the gravity beam to ensure that the movement of the structure is not adversely effected. This gap causes a slight moment to be created which will be resisted through tension in screws, although NZS3603 does not allow for the consideration of combined tension and shear in screws the European code (EN5:2004) suggests the following equations be applied:

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leq 1$$

Where:

$F_{ax,Ed}$ = The axial demand of the screw

$F_{ax,Rd}$ = The axial capacity on the screw

$F_{v,Ed}$ = The shear demand of the screw

$F_{v,Rd}$ = The shear capacity on the screw

This equation was verified for the top screw only as this case would be critical.

5.2 Corbels for Gravity Beam Seating

The use of corbels is common practice for structural engineering applications. Due to the sizable gravity loadings present in the structure gravity bearing was the most appropriate method of transferring the gravity loadings of the structure. Details of the corbel arrangement are shown in Figure 5.2.

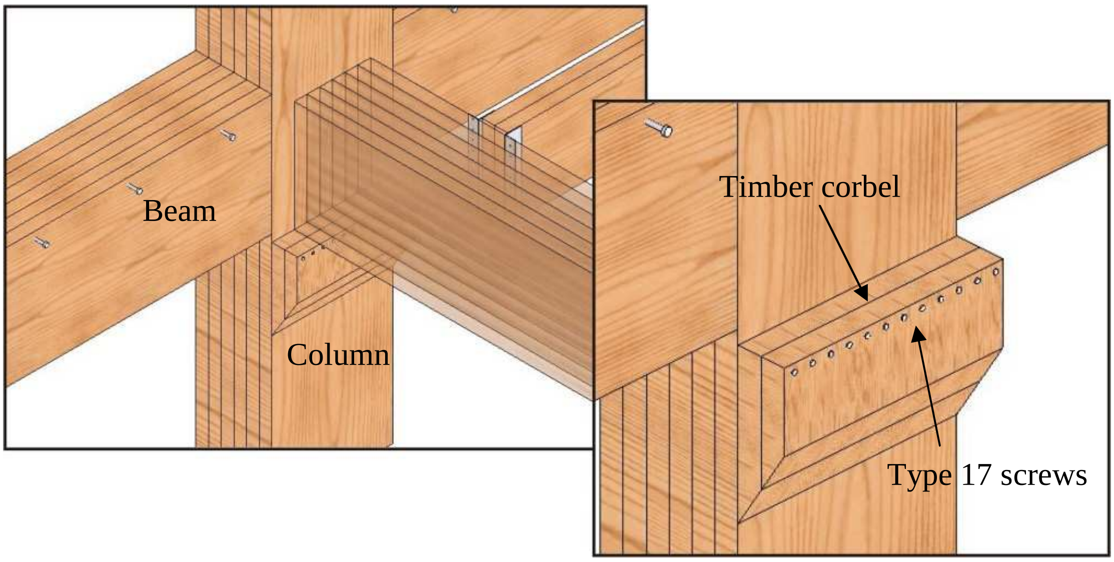


Figure 5.2: Timber gravity corbel

The corbel is glued onto the face of the column during the construction of the member. The seating arrangement will act as a pin support, meaning that only a small amount of moment will be transferred to the gravity frame from the eccentricity of the loading. Figure 5.2 shows the use of Type 17 screws applied near the top of the member. This is necessary due to the poor strength of timber when loaded in tension perpendicular to the grain.

Previous studies into the performance of epoxy dowel connections (Batchelar 2006) have been applied assuming the plane sections remain plane and that traditional elastic theories are applicable. This suggests that the method of transformed sections, common for reinforced concrete, can be applied.

This method involves the calculation of a ratio between the Modulus of Elasticity of the timber and that of the tension steel. As Figure 5.3 shows once the section transformation has been performed force equilibrium can be used to find the neutral axis and the moment capacity is calculated.

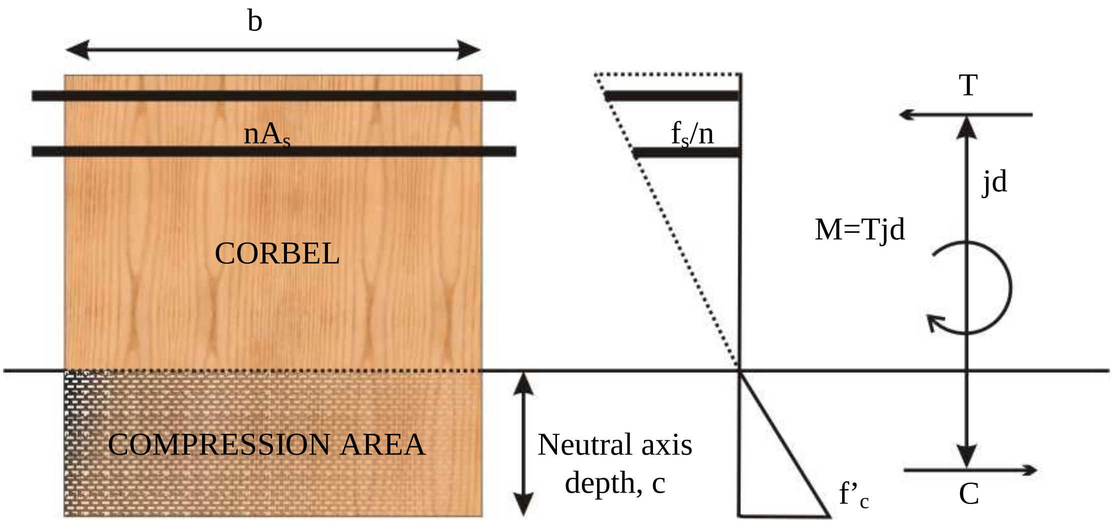


Figure 5.3: Moment calculation using transformed sections

This method can be used to determine the amount of tension strength required in the top of the timber corbel. Once the required tension strength is found the amount of Type 17 screws and the required penetration can be found using characteristic pull out strength of the screw (Gaunt and Penellum 2007).

This seating arrangement was also used for the flooring support for the long span between two structural walls as shown in Figure 5.4.

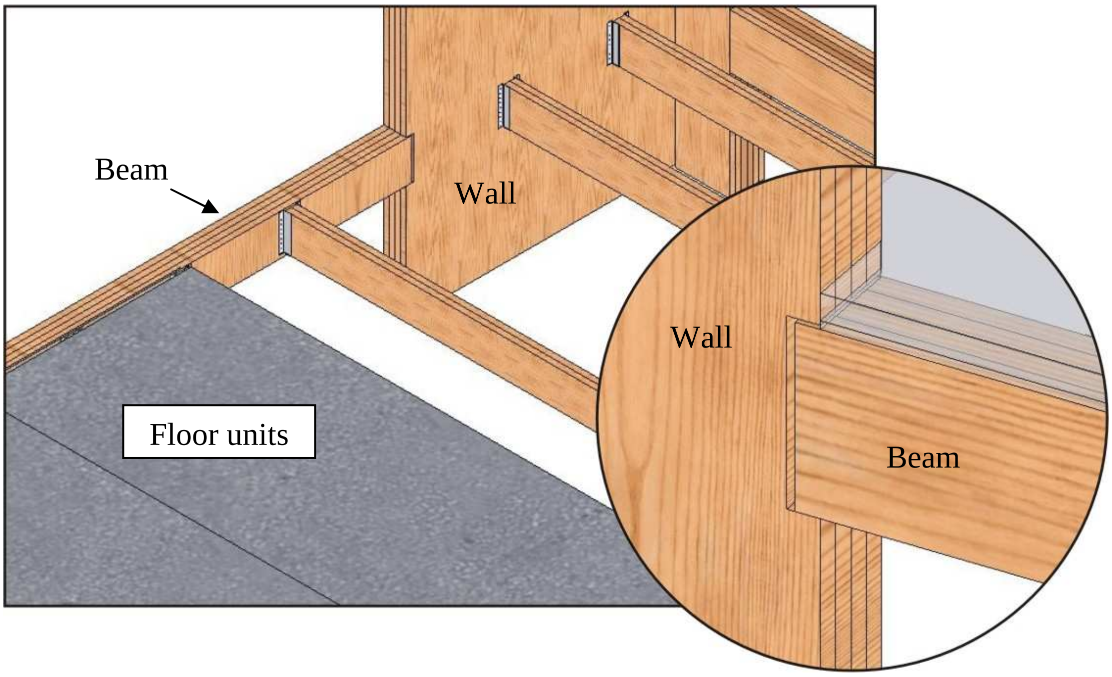


Figure 5.4: Wall corbel seating

During the rocking motion of the walls the timber floor will deform with the shape of the walls and connected gravity beam. Although it is apparent that this will cause damage to the concrete topping, (Figure 5.5) this damage will be minor and easily repaired. A gap will be cut into the flooring unit to allow for the movement between the two closely spaced walls in the system.

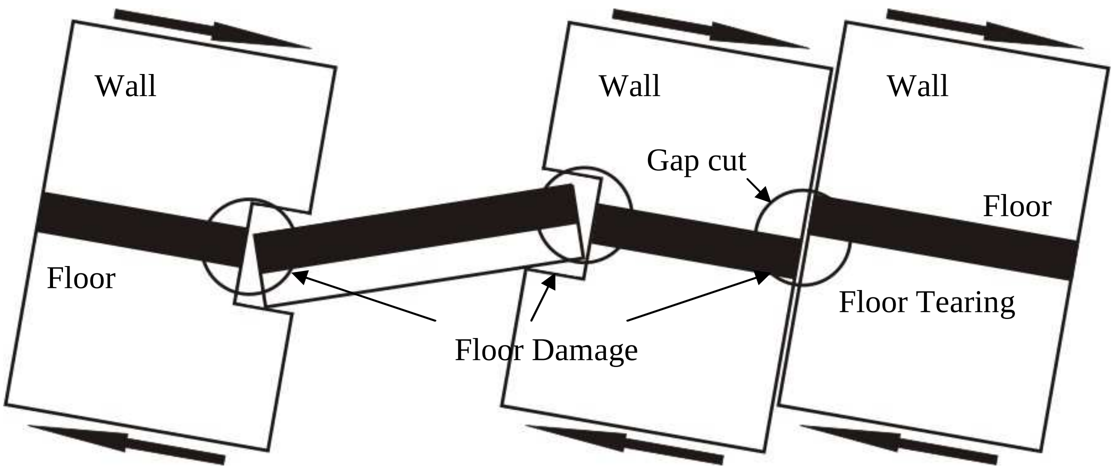


Figure 5.5: Floor damage due to wall lateral displacement

5.3 Seismic Seating

Although it is possible that the friction at the face of the beam to column connection is adequate, the current New Zealand code states that it is not possible to consider this in the transfer of the ultimate gravity loading to the column. It is therefore important to have a corbel at the connection. The major issue with the attachment of these corbels is that they can cause the beam to rise up during rocking leading to increased un-necessary damage to the floor. Several corbels were tested under seismic gravity load as detailed in Chapter 6.

5.4 Wall Foundation Attachment

The energy dissipation of the Hybrid connection described in Chapter 4 is a crucial part of the system performance. The detailing of this connection is discussed in the following paragraphs.

5.4.1 Development of a Fuse Type Dissipater

The concept of fusing a diameter, while providing anti-buckling restraint, for a dissipation device was originally conceived in concrete for the attachment of external dissipation (Marriot 2006). This is formed from a given bar milled down to a certain fuse diameter over a specified length (Figure 5.6a). A steel tube is placed over the fuse length and filled with epoxy to prevent buckling when loaded in compression. This leads to very stable hysteric behaviour under tension and compression loading (Figure 5.6b). In the case of internal attachment of the dissipation device the surrounding cavity inside the wall is used as an anti-buckling restraint.

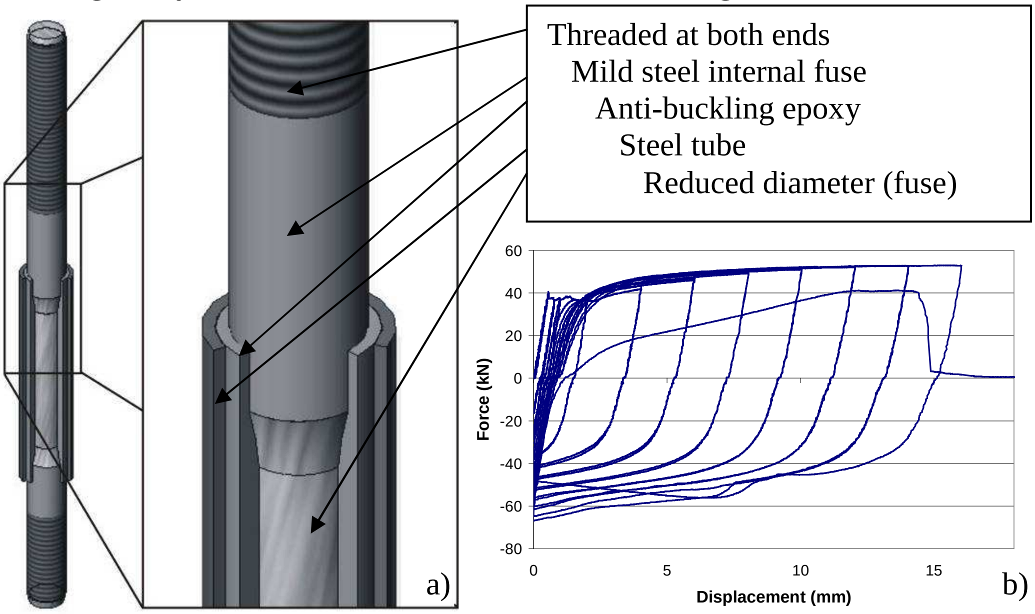


Figure 5.6: a) Fused dissipater with anti-buckling restraint b) Hysteretic behaviour of dissipater

5.4.2 Internal Attachment of Dissipater into Member

This type of dissipater requires a strong attachment to both the foundation and to the member. Internal attachment of these dissipation devices has been performed successfully using epoxy (Palermo et al. 2005, 2006). The design procedure of this attachment is a slight modification of that suggested in the Timber Design Guide (Buchanan 2007) which is as follows:

> An epoxy bonded steel connection loaded in axial tension will satisfy the following equation:
>
> $$N^* \leq \phi Q_n$$
>
> Where:
>
> N^* = Design axial force procedure by the factored design loads (kN)
>
> □Q_n = Design strength of the connection (kN); which will be taken as the least of equations 1, 2 and 3 below
>
> □ = Strength reduction factor
>
> Q_n = Nominal axial strength of the connection (kN)
>
> The design axial strength considering the steel in yielding is:
>
> $$(\phi Q_n)_{steel} = \phi_{steel} n A_s f_y \qquad (1)$$
>
> Where:
>
> □$_{steel}$ = 0.8 (NZS 3404:1997 for steel members in tension)
>
> n = Number of steel bars
>
> A_s = Cross section of each bar (mm^2/1000)
>
> f_y = Characteristic yield strength of steel (MPa)
>
> The design strength considering the wood fracture at the end of the bar is:

$$\left(\phi Q_n\right)_{wood} = \phi_{conn} k_1 A_w f_t \tag{2}$$

Where:

ϕ_{conn} = 0.7 (a timber connection other than nails or bolts)

k_1 = Duration of load factor

A_w = Net area of wood cross section, excluding drilled holes (mm^2)

f_t = Characteristic tensile strength (Mpa)

The design axial capacity of a bar in pullout is:

$$\left(\phi Q_n\right)_{pullout} = \phi_{conn} k_1 n k_g Q_k \tag{3}$$

Where:

k_g = Bar group reduction factor (1.0 for 2 bars; 0.9 for 3 or 4 bars; 0.8 for 5 or 6 bars) (Korin et al. 1999)

$$Q_k = 6.73 k_b k_e k_m \left(l/d\right)^{0.86} \left(d/20\right)^{1.62} \left(h/d\right)^{0.5} \left(e/d\right)^{0.5} \tag{4}$$

Where:

d = Steel bar diameter

l = Embedment length

h = Hole diameter

e = Edge distance from the centre of the bar

k_b = Bar type factor (threaded: 1, deformed: 0.8)

k_e = Epoxy factor (Based on epoxy brand used)

k_m = Moisture factor (moisture content <15%: 1.0, 15-22%:0.8)

The above equation (4) has been derived from a large amount of testing performed at the University of Canterbury on epoxy steel rods embedded parallel to the grain in Glulam timber (Deng 1997) and is purely empirical. A similar equation (Van Houtte 2003) has been proposed for epoxy steel rods embedded parallel to the grain in LVL:

$$Q_k = k\left\{\left(1.885\times10^{-4}\right)h^2 l f_s + 15\right\} \tag{5}$$

Where:

k = 0.85 reduction factor

f_s = LVL shearing stress (Mpa)

This equation has been verified to be accurate for embedment lengths 50 – 400mm (Van Houtte 2003) however due to the lack of further information this has been applied to the connections above this length. This equation was used to calculate the embedment length required for the bonded length of the dissipater.

Given these recommendations the following design procedure was devised:

Calculate the amount of dissipation required in the system using the hybrid design procedure outlined in Chapter 4. From this calculate the amount of steel required using the following equation:

$$d = \sqrt{\frac{4N^*}{\phi\pi n f_y}} \tag{6}$$

Where:

N^* = Design axial force

n = Number of bars

f_y = Bar yield strength

ϕ = 0.9

Once diameter and number of bars to be used has been set the over-strength of a single bar is calculated:

$$N^*_{O,S} = \phi_O A_s f_y \tag{7}$$

Where:

$N^*_{O,S}$ = The over strength axial force of a bar

ϕ_O = 1.3 Over strength factor

It is suggested that in order to control the yield point of the bar that the diameter of the remainder of the bar be 1.25 larger than that of the fuse. Therefore, the diameter of the bonded section, D, is equal to 1.25d, the diameter of the un-bonded section. Once this diameter is set the embedment length of the bar is determined using the Van Houtte formula to ensure that the bonded bar strength is less than the over strength of the fused bar.

$$l = \left(\frac{N^*_O}{\phi_{conn} k_1 k_g k} - 15 \right) \Big/ 1.885 \times 10^{-4} h^2 f_s \qquad (8)$$

Where:

ϕ_{conn} = The connection reduction factor (0.7)

It is suggested that for the bars placed in the wall that a stagger be used (Buchanan 2007) this will cause the largest amount of timber possible to be in tension ensuring that a group failure of the connection does not occur. These bars should be staggered by at least 75mm to ensure that stress concentrations do not occur.

This bar is then grouted into the foundation at a length adhering to the New Zealand Concrete code (NZS3101) basic development length equation:

$$L_{db} = \frac{0.5 \alpha_a f_y}{\sqrt{f'_c}} d_b$$

Where:

f_y = The yield strength of steel bar

f'_c = The compression strength of the concete

α_a = 1.3 (more than 300mm of concrete is cast below the bar)

d_b = Diameter of the bar

5.5 Column Foundation Attachment

An issue arose during the detailing of the building regarding the attachment of the columns epoxied bars into the foundation. The 25mm diameter bars required a considerable development length causing an increase in the depth of the foundation.

Several options were discussed, however a few major restriction limit the options available:

- Due to quality control issues it is not desirable to use epoxy resin on site
- The design option must not increase the depth of the foundation
- The attachment must remain elastic while ensuring the yielding of the dissipater

These restriction lead to the development of a steel shoe which will be attached to the base of the column as shown in Figure 5.7.

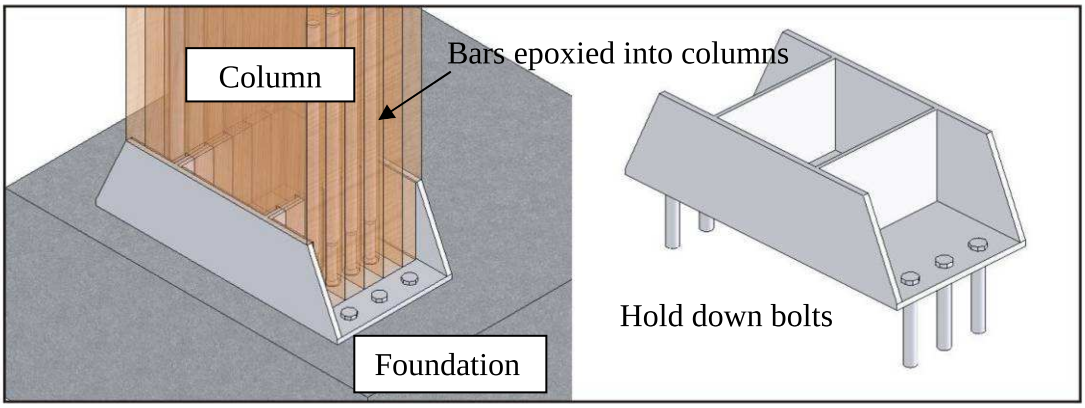


Figure 5.7: Column base shoe attachment

As shown above the epoxied bar dissipaters are butt welded to the steel plate and the plate is then bolted to the foundation. In order to gain the required bending strength in the section vertical steel plates are added. It is important to ensure adequate clearance is left around the specimen so that the controlled rocking motion is not hindered. The plate will be bolted to the foundation using high strength bolts.

5.6 Shear Transfer from the Slab into the Wall

One of the largest challenges in the buildings design is the method of shear transfer from the flooring units into the walls. The timber-concrete composite flooring system relies on topping concrete to achieve diaphragm action for the building. This assumption can be considered to be a however it reduces the importance of the connection between each prefabricated panel.

In designing the shear connection for the wall unit, the method of transfer used in reinforced concrete buildings was considered. From this the detail shown in Figure 5.8 was designed.

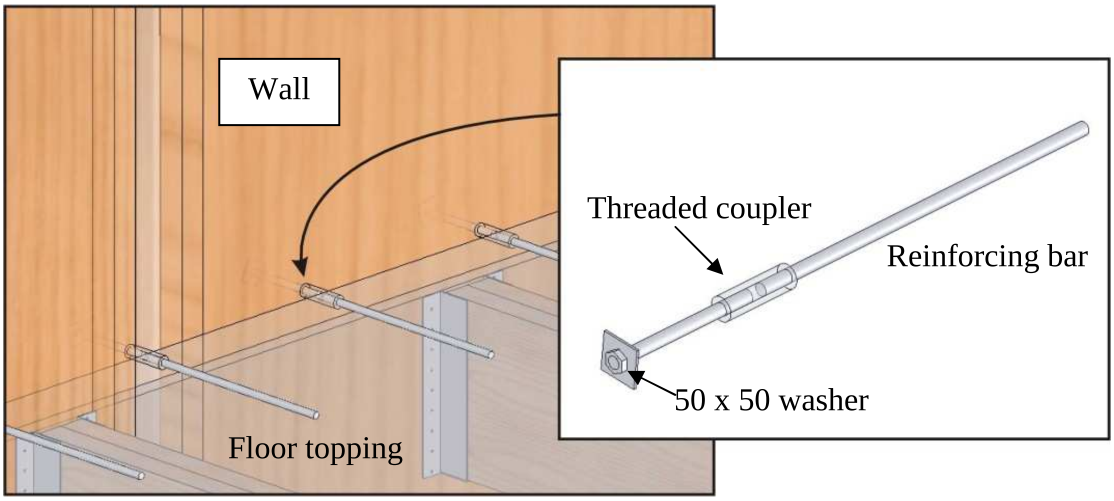


Figure 5.8: Wall shear transfer connection

The design of this bar will be carried out using a modified version of Johansson's yield line theory (EN5 5:2004). It will follow the design set out for a fastener in a rigid medium and the failure modes shown in Figure 5.9 will be assumed to occur.

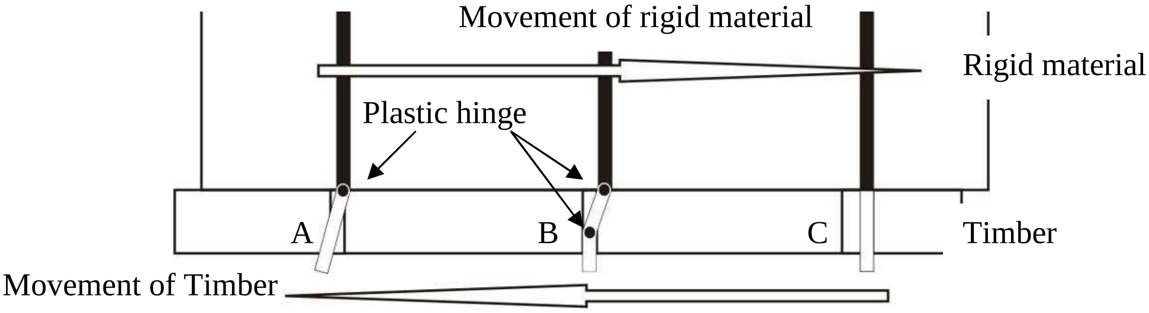


Figure 5.9: Shear fastener failure mechanisms

The strength of a single fastener will be designed calculated using the following equations (A, B, and C respectively):

$$F_{v,Rk} = \min \begin{Bmatrix} f_{h,k} t_1 d \left[\sqrt{2 + \dfrac{4M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right] + \dfrac{F_{ax,Rk}}{4} \\ 2.3\sqrt{M_{y,Rk} f_{h,k} d} + \dfrac{F_{ax,Rk}}{4} \\ f_{h,k} t_1 d \end{Bmatrix}$$

Where:

$f_{h,k}$ = Characteristic bearing strength in the timber member

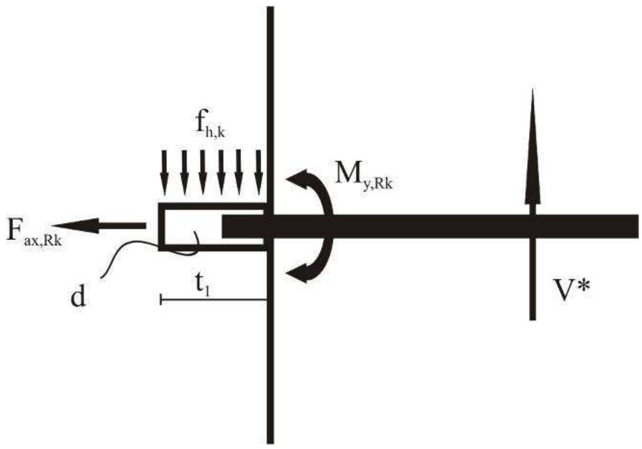
t_1 = Thickness of timber member

d = Diameter of the fastener

$M_{y,Rk}$ = Characteristic fastener yield moment

$F_{ax,Rk}$ = Characteristic withdraw of fastener

Several simplifications are made to make this applicable to this type of connection. These simplifications are shown in Figure 5.10.



$f_{h,k}$ = Compression strength perpendicular to the grain

t_1 = Length of TCM

d = TCM diameter

$M_{y,Rk}$ = Yield moment of reinforcing drag bar

$F_{ax,Rk}$ = Axial withdrawal of the bolt

Figure 5.10: Simplifications of Johansson’s yield line theory

5.7 Shear Transfer for the Slab into the Frame

As the length of the area that the frame system in contact with the diaphragm is longer than that of the wall, a connection of lower strength that is simpler to attach is used. This connection is simply a coach screw attached to the side of the beam which is then cast into the topping (shown in Figure 5.11).

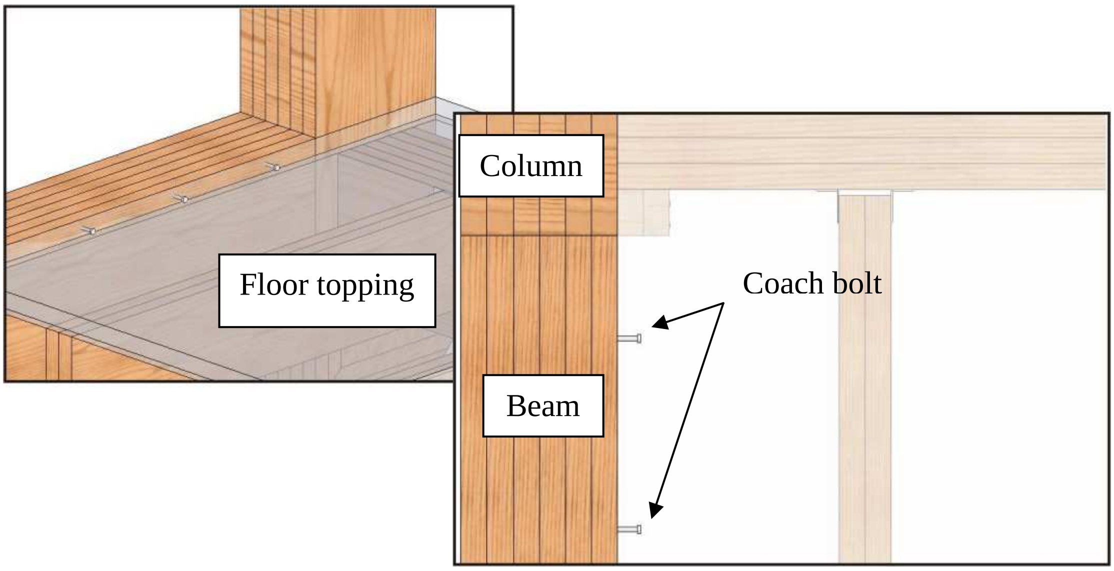


Figure 5.11: Frame shear connection

The design of this connection is carried out using a characteristic value derived from the testing of Siebold (2004). The results for the first 5mm of horizontal displacement is shown in Figure 5.12, the inset shows the behaviour over 80mm of displacement. Note that element 13 and 14 refer to two identical test specimens.

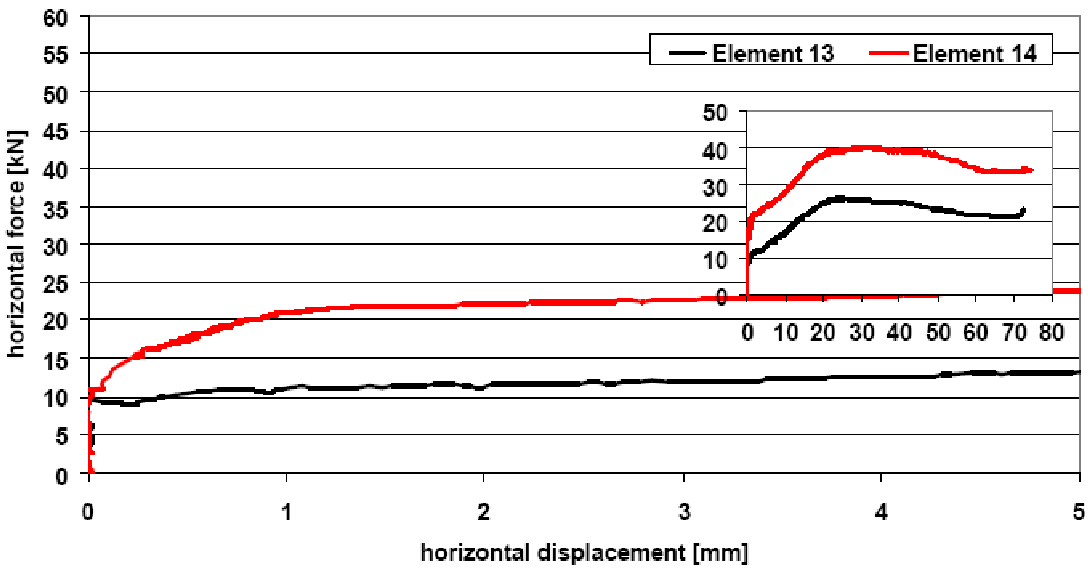


Figure 5.12: Shear test for a single coach screw (Siebold 2004)

As seen above the characteristic strength for a single screw can be taken to be 9.7kN. However, this testing is not completely adequate for this application and further testing of this connection was carried out and is discussed in Chapter 6.

Once this characteristic strength was chosen the following formula is applied for the joint strength:

$$\phi Q_n = \phi k_1 Q_n$$

Where:

Φ = 0.7 for ‘other types of fasteners’ NZS 3603

k_1 = Load duration factor (= 1 for earthquake)

Q_n = Characteristic strength of fastener

5.8 Further Connection Details

Although the above connection details were used for the design of the six storey case study, during the design process several other options were consided. These further options are outlined below.

5.8.1 Corbel for Joist Seating

The main issue with the use of the joist hangers is that they must be attached in an exact location meaning that this attachment must also occur on site. One possible way to negate this situation is the use of a corbel seating as shown in Figure 5.13.

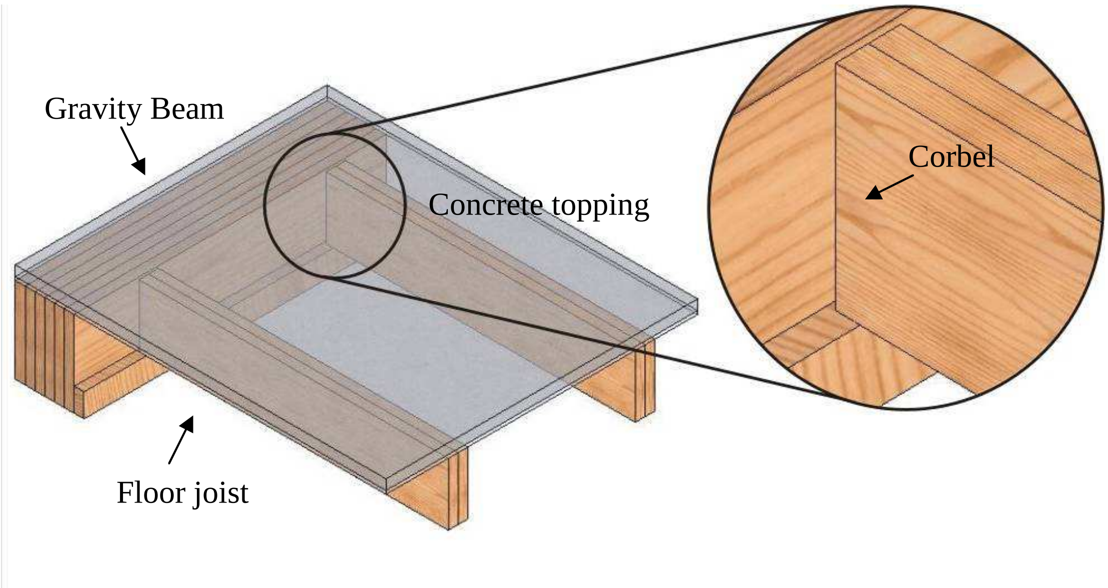


Figure 5.13: Corbel seating of flooring joists

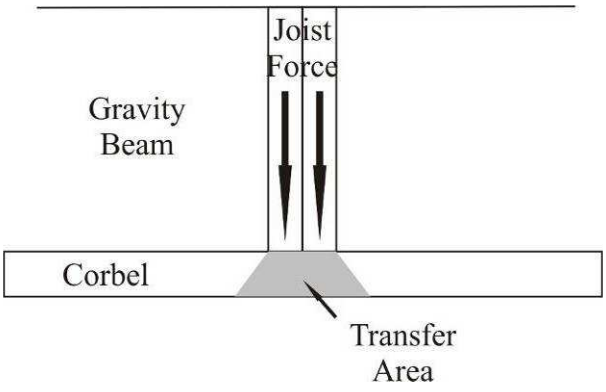


Figure 5.14: Vertical load path

These corbels can be attached as the beam is manufactured and the joist will simply sit in on site. However, the required load path of this attachment may cause problems with this system. As the floor joists span a large distance, the gravity loading required is considerable. This means that the transfer area of the vertical force (shown in Figure 5.14) must also be of a considerable size. As the joists are also of a substantial depth this may cause an unnecessary increase in the depth of the gravity beam.

5.8.2 Use of U Shaped Plate for Energy Dissipation

The use of U shaped flexural plates (UFP's) for energy dissipation (Figure 5.15) was first proposed by Kelly et al. in 1972. Later, during the PRESSS testing programme (Conley et al. 2002) this element was used with great success when coupling two rocking wall members. Testing at the University of Canterbury has shown that similar results can be achieved when the UFP's are used to couple two LVL walls (Iqbal et al. 2007).

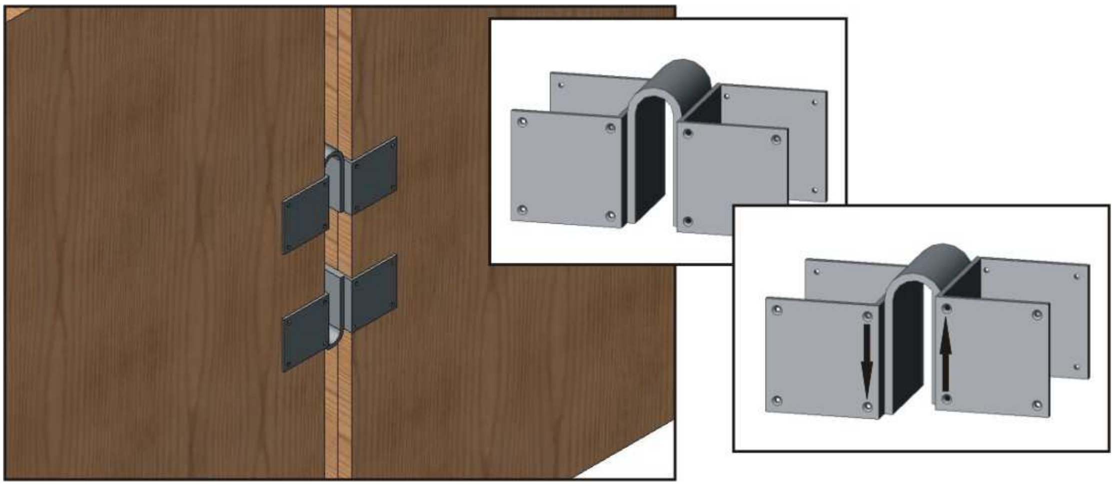


Figure 5.15 U shaped Flexural Plate (UFP)

Although this method of dissipation was not utilised in the case study building, this element has some significant advantages as it does not require the use of epoxy. Further to this the use of the axial dissipation devices described in Section 5.3.1 do not perform well under significant compressive displacement, meaning the UFP's are more suited to coupling the walls.

5.8.3 Coupling Walls with Plywood Sheets

Testing performed at the University of Canterbury has also investigated the use of nailed plywood for the coupling of wall members (Smith et al. 2007). These sheets were attached to each face of the wall using nailed perimeters. The nail spacing was then altered changing the amount of hysteretic dissipation in the system. The result of this testing is shown in Figure 5.16.

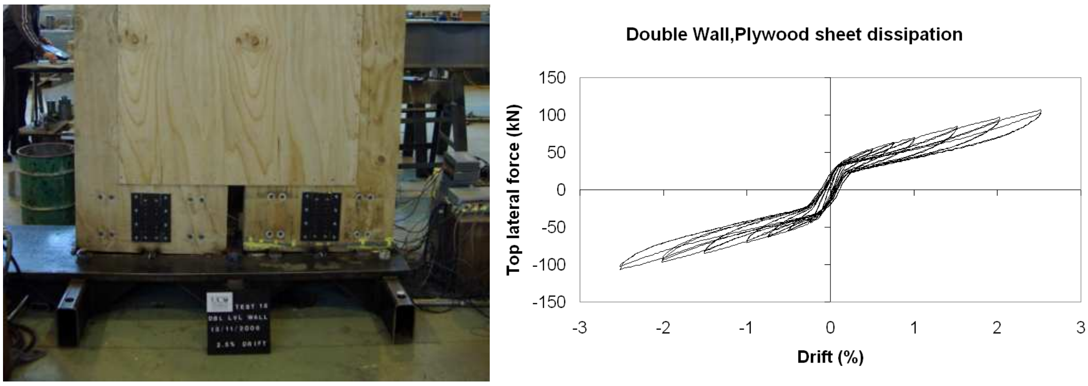


Figure 5.16: Wall testing coupled with nailed plywood (Smith et al. 2007)

It can be seen in Figure 5.16 that the characteristic pinching behaviour of nailed plywood is evident. This causes the damping of the system to decrease during cyclic behaviour. Although a decrease in damping does occur, this system has several advantages as it is cheap and easily placed on site.

6 Connection Testing

The following chapter outlines tests performed to assess the performance of key connections adopted in the design of the timber case study building. The first tests assess methods to resist shear at the base of a wall or column member due to lateral loading. Secondly a series of four simple pushout tests is performed to find an initial indication of characteristic strength of the beam to floor diaphragm connection suggested in Section 5.7.

A beam to column subassembly was then used to investigate the interaction of key factors in the systems performance. A series of test were performed aiming to answer the following questions:

- What is the effect of placing steel armouring on the column face in the beam to column connection?
- What is the effect of altering the initial post tensioning force in the tendon on moment response of the connection?
- How accurately does the design procedure outlined in Newcombe et al. 2008 predict the key characteristics (Moment response, tendon force, and neutral axis depth) during lateral movement of the column?
- What effect does the placement of corbels on the column face under the beam have when a shear load is placed on the beam?
- What effect does the placement of a timber-concrete composite floor unit have on the moment response of the beam to column connection?

6.1 Testing of Angle Shear Keys

During the testing of the single LVL hybrid wall (Smith 2006b) a major issue relating to the attachment of shear keys to resist seismic shear at the base of the wall was recognised. This will be an issue for both wall to foundation and column to foundation members. It was originally stated that the use of circular shear keys is preferable to the stiffer angular shear key previously used as movement and rolling over the keys is allowed. Although this is adequate for short term seismic testing it is necessary to revisit this idea if serviceable loading is to be considered. Due to the stress concentrations created on the end of the member (Figure 6.1a) considerable damage occurs over the long term (Figure 6.1b).

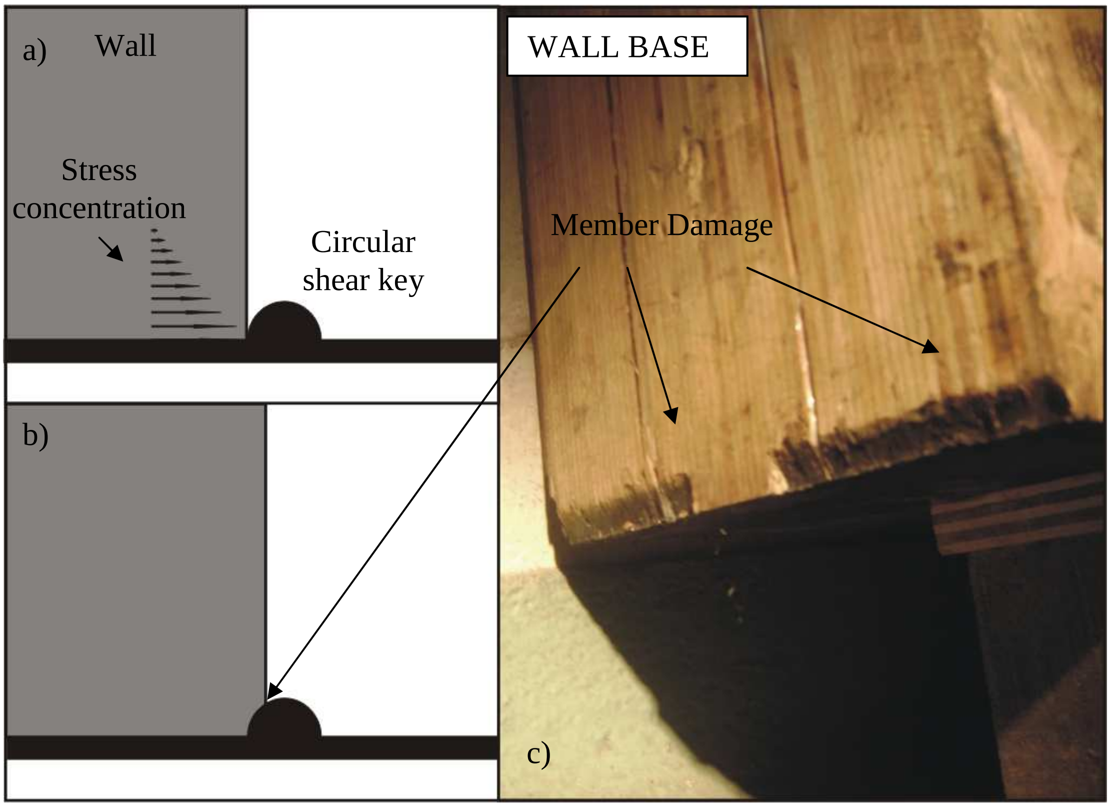
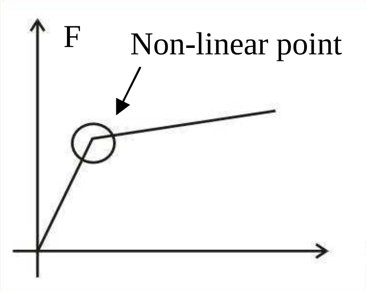
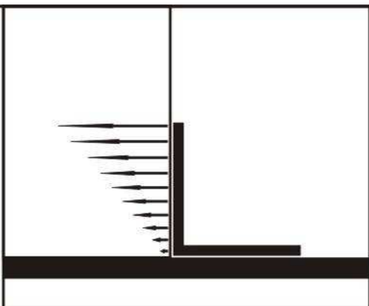
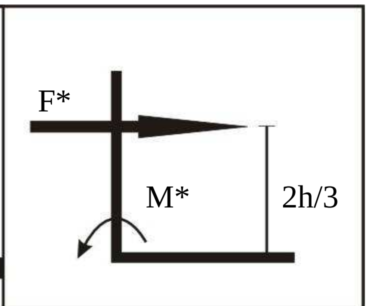


Figure 6.1:a) Stress concentration at base of member b) Damage to member c) Damage in base of wall member after testing

The design of the half circle shear keys was undertaken considering the need for constant seismic testing in which it is important that the shear key does not hinder the movement of the column or wall under a continued rocking movement. It is therefore proposed that a different design be use for the actual connection as it is unlikely that this continued rocking motion will occur frequently during the structures lifetime.

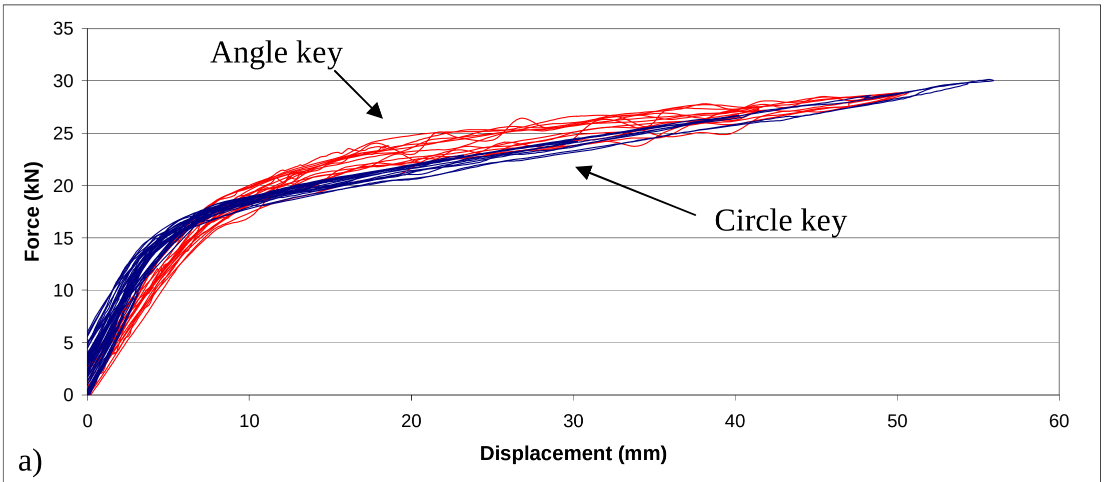
The new design consists of a simple piece of angled steel, designed to allow the rocking of the column through yielding. The design procedure of the key is laid out in Table 6.1.

Table 6.1: Design of foundation shear key

		
Non-linear point predicted	Triangular force distribution	Angle designed to yield at this force (at θ_y disp)

The non-linear point and corresponding force is found from the predicted response of the section. As shown, a triangular distribution is assumed as the member begins to rock. The calculation of the non-linear force and the assumed distribution means that the thickness of the angle needed to ensure yield can be predicted.

Some simple seismic testing on column to foundation joint was carried out in order to validate this design procedure which is shown in Figure 6.2.



a)

		
b) -2.5% Drift	0.0% Drift	2.5% Drift

Figure 6.2: a) Force versus Displacement with angled and circle shear key b) Angle during testing

Figure 6.2a compares the moment versus drift response to that of the half circular key in which movement was considered to be un-hindered and it is clear that the difference is negligible. Further to this, due to the larger surface area of the angled shear key the damage to the base of the wall or column shown in Figure 6.1 will not occur. Therefore, if the angled shear key is designed properly, it will not hinder the rocking of the column or cause damage to the member.

6.2 Pushout Testing of Floor to Beam Shear Connection

One of the crucial connections in the any building is that of the floor diaphragm into the seismic resisting system. In most seismic designs it is assumed that the floor diaphragm acts as a rigid block and that the connection between this floor and the frames and walls always remains elastic. Initial designs of the connection between the flooring and the seismic frames used the empirical values found by Siebold (2004), however, these tests were performed with a coach screw placed in the top of the member as shown in Figure 6.3a. The method of connection used in this design has the screw placed parallel or perpendicular to the grain as shown in Figure 6.3b.

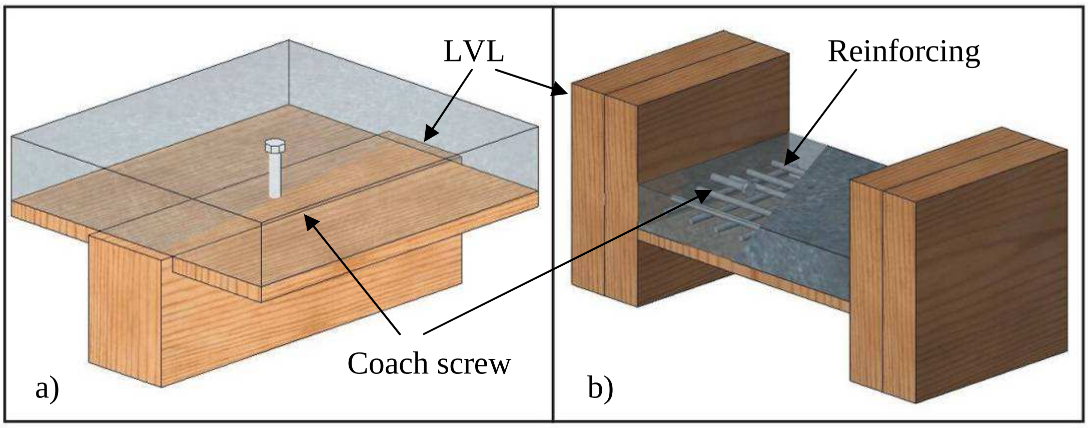


Figure 6.3: a) Siebold pushout specimen b) Test Specimen used

In order to discover if the numbers used in the design are adequate a series of simple pushout tests was used. As shown in Figure 6.4 this pushout test only found the direct shear strength of the connection. Two blocks of LVL were used on either side with a 65mm concrete topping with 10mm bars at 50mm c/c (shown in Figure 6.4). Two 150mm long coach screws of 16mm diameter are embedded in the face of the blocks. The top of the head of the screw is left protruding 75mm into the concrete topping.

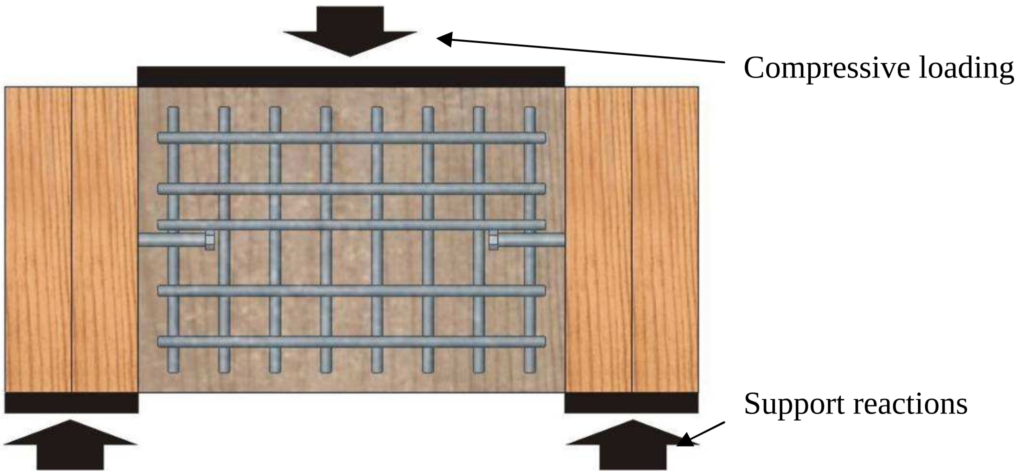
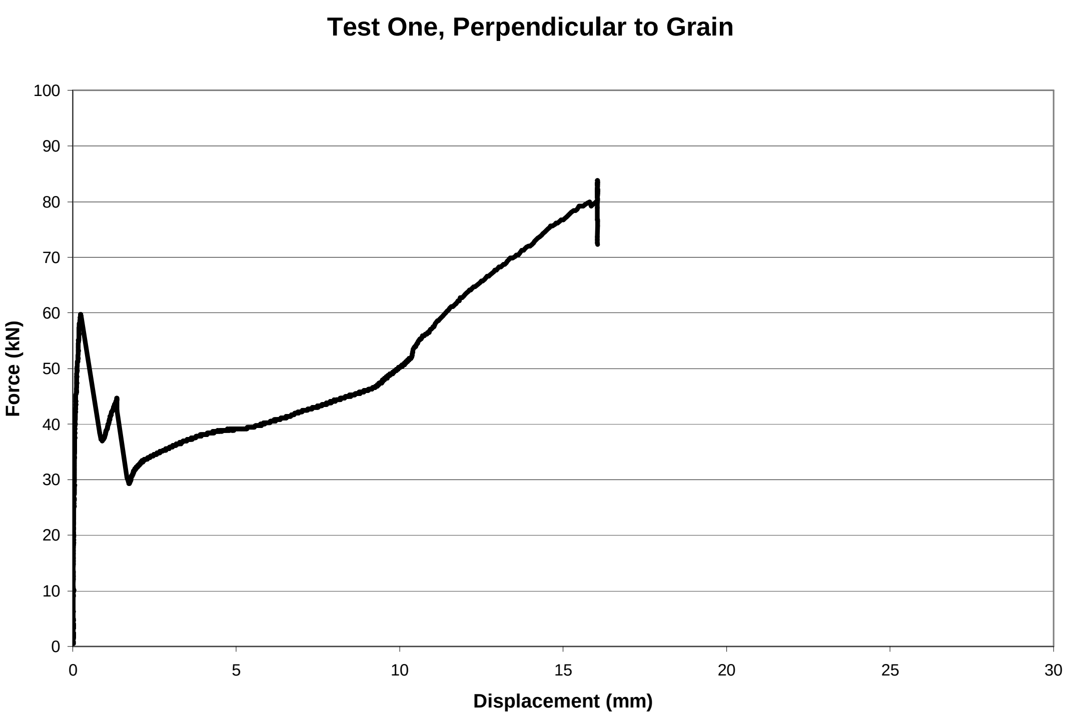


Figure 6.4: Application of load to pushout specimen

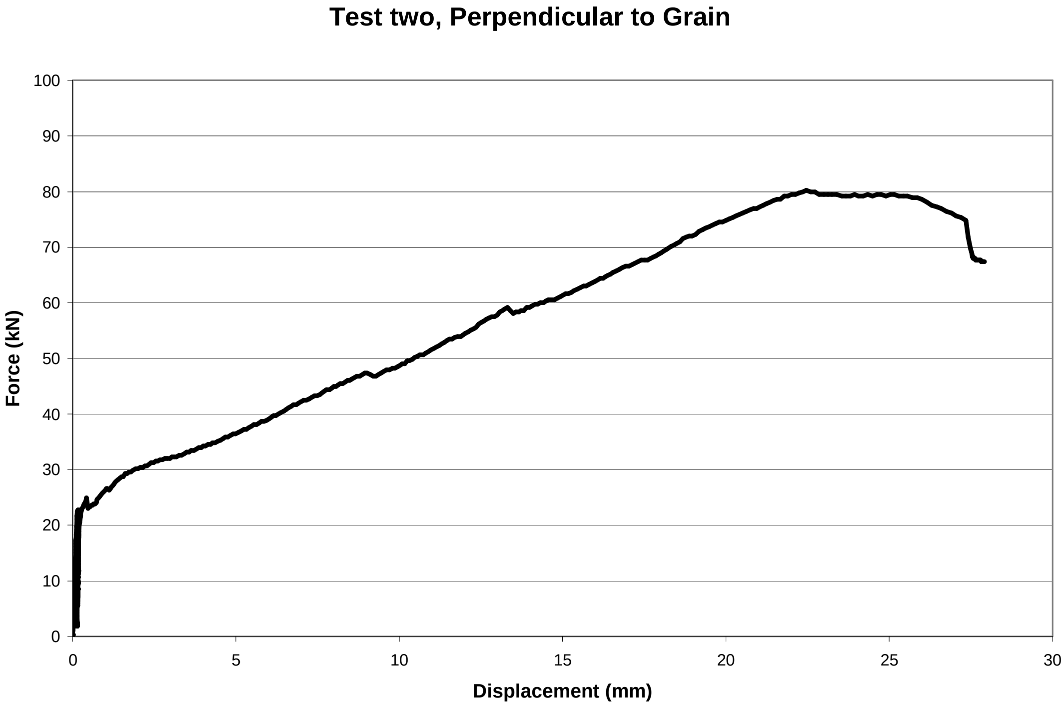
Four tests were carried out using this configuration: Two with the grain running perpendicular to the applied force and two with the grain running parallel.

6.2.1 Perpendicular to Grain Testing

The results from these two pushout test are shown below in Figure 6.5, note that these results are for the push out specimen (i.e for two coach screws).



a) Specimen one



b) Specimen two

Figure 6.5: Pushout shear testing perpendicular to the grain

In Figure 6.5, although the two tests were set up in the same manner two different failure modes occurred. The first test displayed a sudden slipping of the screw characterised by the two spikes in the force displacement graph. These spikes were accompanied buy a cracking noise from the timber. This indicates a sudden friction failure occurring at the interface between the topping and the LVL. Once this occurs ductile behaviour is exhibited. The second test did not display the sudden friction failure of the first. Progressive cracking appeared in the topping and propagated until total failure. Although two different failures occurred it can be seen above that the same peak strength is reached in both cases. From inspection of the test specimen it is likely that this is the point of concrete failure shown in Figure 6.6a.

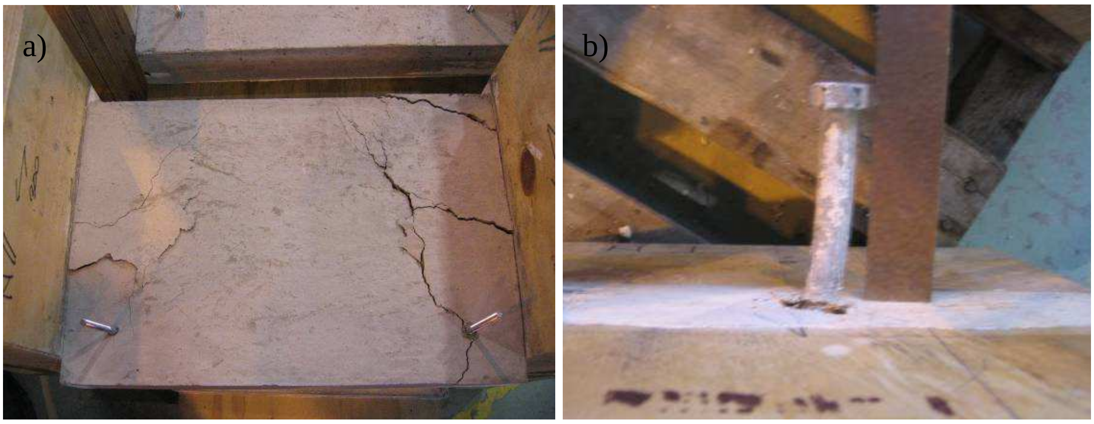
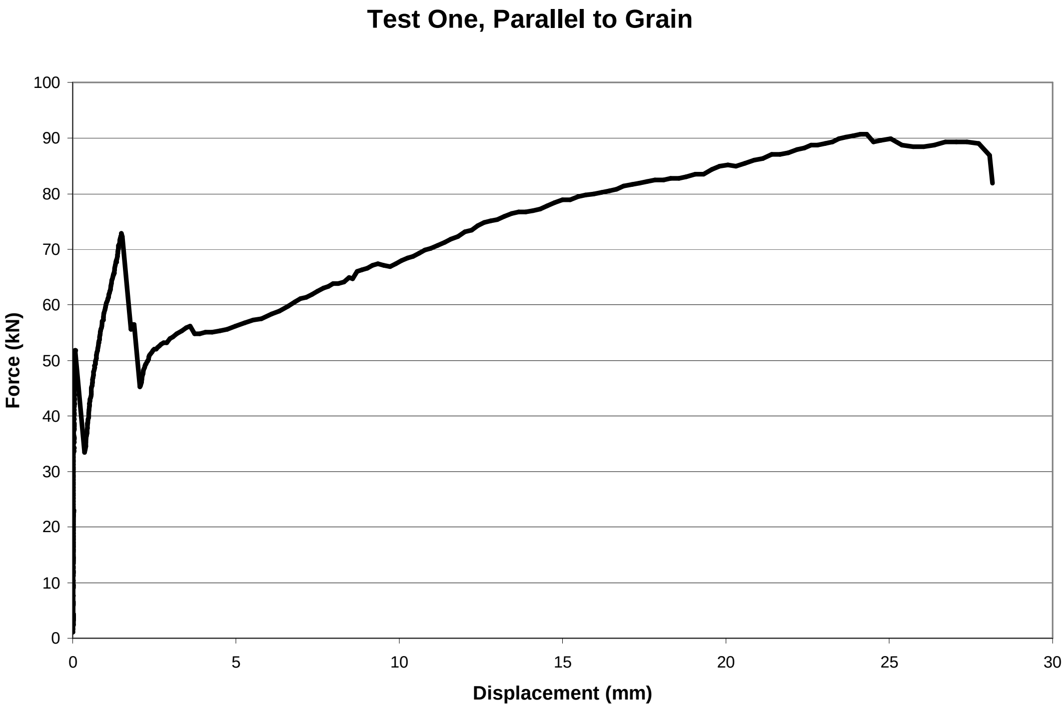


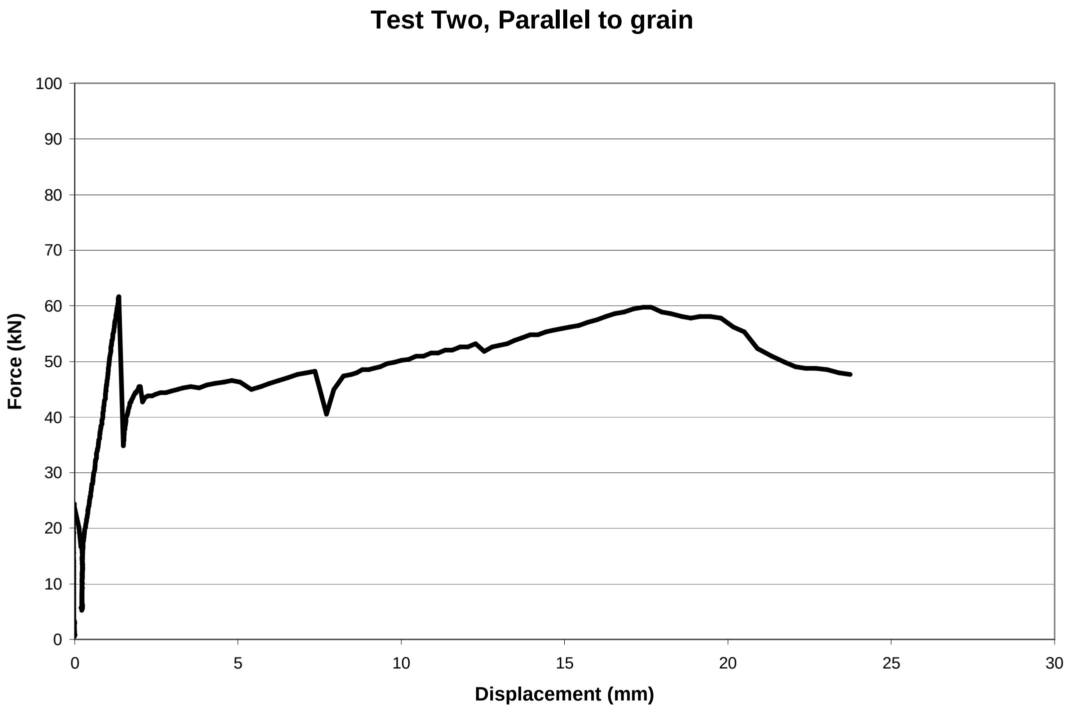
Figure 6.6: Pushout specimen after testing: a) Damage to the concrete b) Deformation of coach screw inside concrete

6.2.2 Parallel to Grain Testing

The results from these two pushout test are shown below in Figure 6.7.



a) Specimen one



b) Specimen two

Figure 6.7: Pushout shear testing parallel to the grain

Both of these tests displayed the sudden shear failure, and again this failure was accompanied by a large cracking noise. Test one displayed the same level of ultimate strength as that of the two parallel tests, however, the second test displayed significantly lower ultimate strength. Visual inspection of the test indicated that local crushing of the concrete around the coach screw due to inadequate cover may have been responsible for this significantly lower strength. The failure mode of the two specimens (Siebold and modified) is compared in Figure 6.8.

a) Failure of specimen (Siebold 2004) b) Failure of modified specimen

Figure 6.8: Failure of push-out specimens

It can be seen from the pictures in Figure 6.8b that when the failure mode of this connection occurs, it is very different to that of the Seibold test (shown in Figure 6.8a). It is clear from these tests that the failure mechanism is in the timber with splitting of the block in the direction of the veneer. The failure of the modified connection clearly occurs in the concrete which cracks and a clear shear failure can be seen in the coach screw. The slip and ultimate points are listed in Table 6.2.

Table 6.2: Significant values from pushout testing

	Test One Perp	Test Two Perp	Test One Para	Test Two Para
At slip	59.5 kN	23.0 kN	51.5 kN	24.4 kN
Ultimate	79.7 kN	80.3 kN	90.7 kN	59.7 kN

As shown in Table 6.2 the results did not display consistent values. However, some common patterns are apparent:

- A minimum characteristic strength of 10kN per screw is suggested.
- Failure occurs in the concrete topping not in the LVL
- Perpendicular to grain behaviour is not different from the parallel to grain behaviour
- Friction failure may occur suddenly
- An ultimate strength of approximately 80 kN can be expected regardless of initial failure method provided adequate cover is used
- Ductile behaviour is provided by the coach screw

Although these results are useful in understanding the failure mode of the connection, a significant amount of testing should be performed to fully understand the performance of this connection.

6.3 Testing of Beam to Column Connection using Differing Interfaces

During the design of the beam to column connections for the case study timber building it was decided to armour the column face in order to eliminate the weakening effect that the compression perpendicular to the grain has on the overall strength of the connection. Although this effect has been speculated (Newcombe 2008) it has never been specifically tested.

In order to investigate this effect a series of tests was devised using a 2/3 scale model of a LVL internal beam to column connection. This represents the corner column of a five storey building with a bay length of approximately 4.5 metres. The interstorey height is 3m giving a total scaled column height of 2m. A constant axial force of 56kN is applied representing the gravity loading from the above floors. A single 7 wire tendon is placed in the centre of the beam. The column member is 338mm in depth and 171 mm in width. The beam member is 300mm deep and 171 wide. The test set up is shown in Figure 6.9.

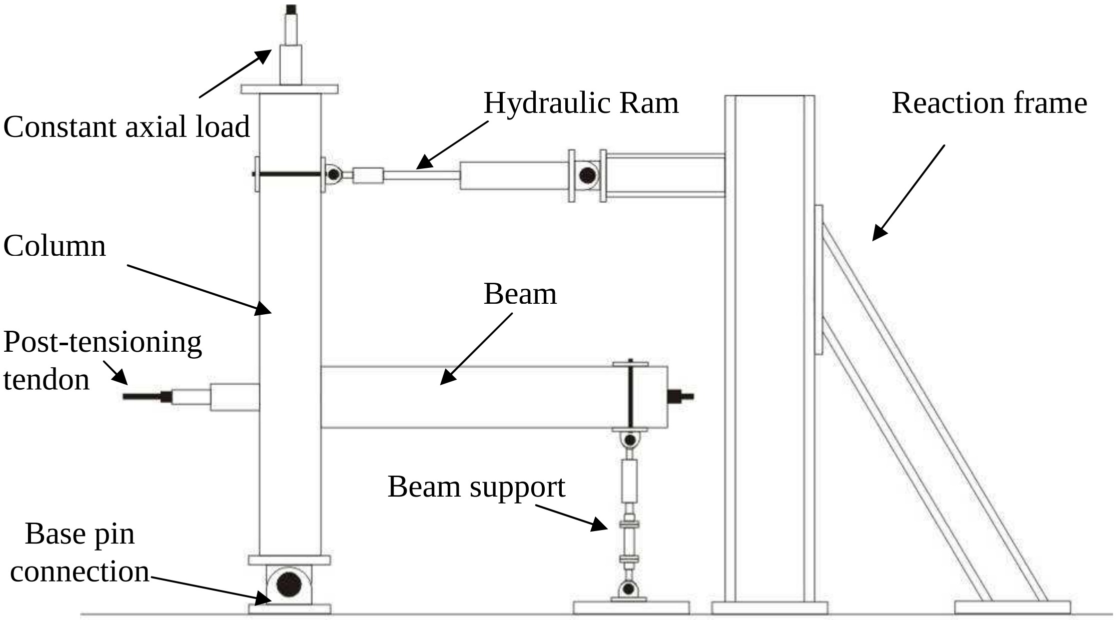


Figure 6.9: Beam to column test setup

This beam to column connection has been designed to withstand many cycles with the connection remaining elastic, avoiding significant inelastic compression of the LVL. This means the ratio between the connection capacity and the beam strength and column strength is not optimised.

6.3.1 Loading Protocol

The beam to column joint was tested under quasi-static loading using a modified version of the ACI code protocol (Figure 6.10), ACI T1.1-01 and ACI T1.1R-01 (2001). For each value of drift, two cycles are applied to the subassembly before a higher level of drift is applied. The loading rate was set at 2 seconds between increments ensuring that little or no dynamic effects occur. The displacement procedure is uploaded to the computerised test controller.

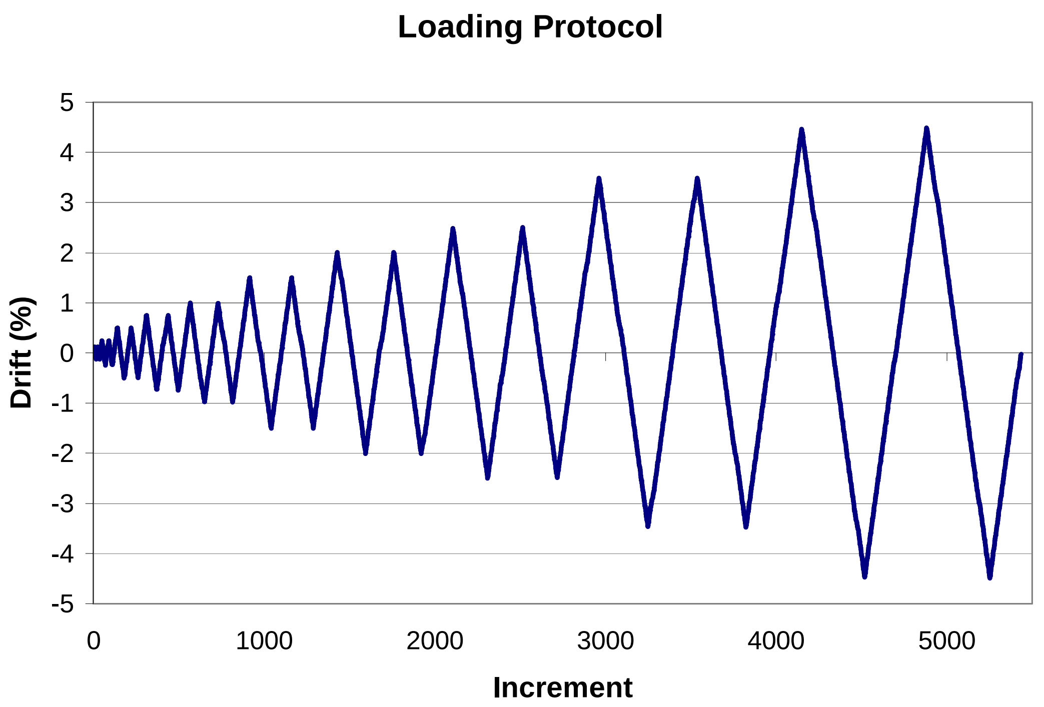


Figure 6.10: Plot of the loading protocol used

6.3.2 Testing Results

The first three tests were performed with the application of a 10mm steel plate between the face of the column and the end of the beam as shown in Figure 6.11.

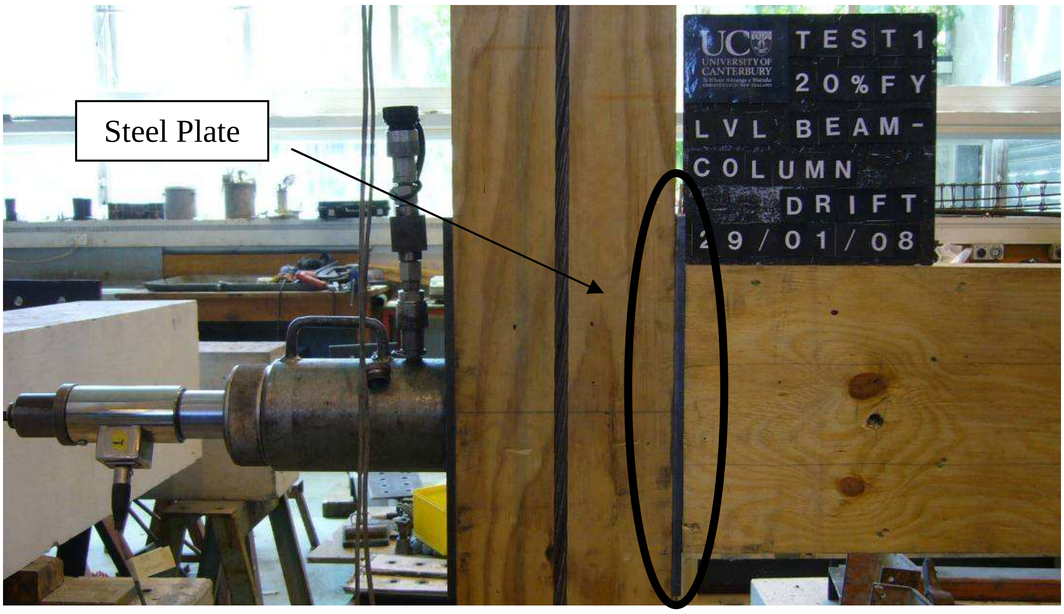
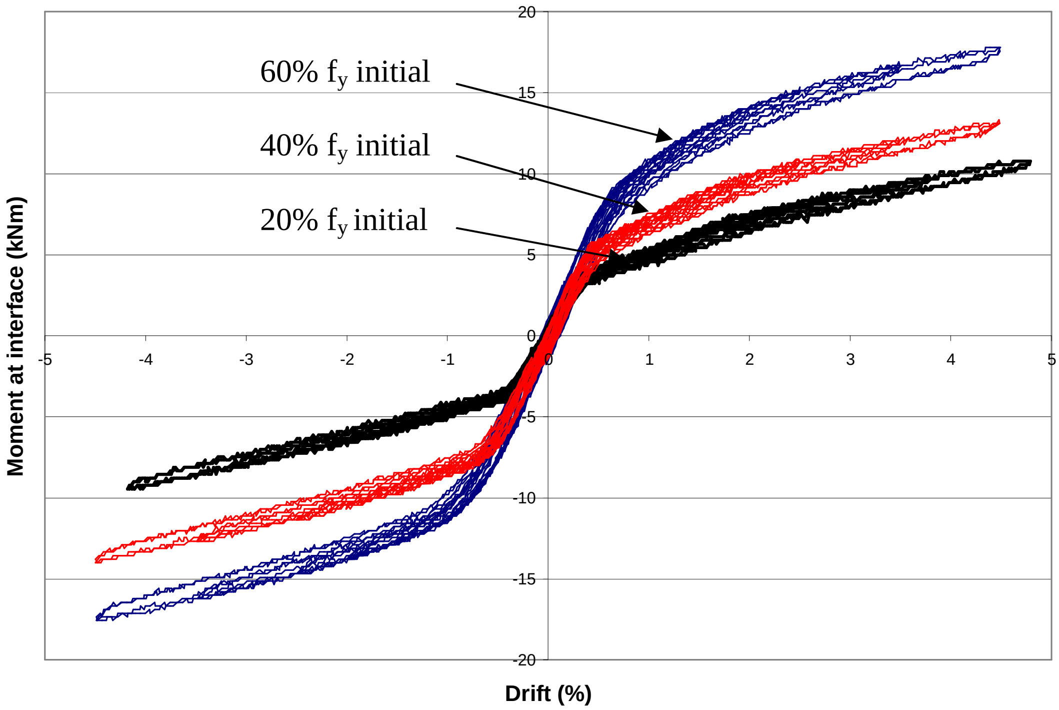


Figure 6.11: Steel plate attached to the face of the column

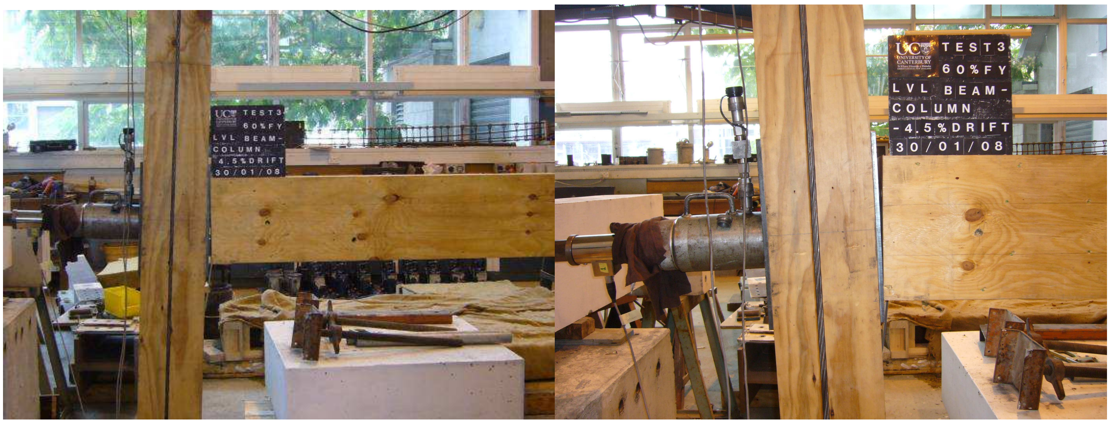
These tests were performed with three different levels of initial post tensioning; the results of these tests are shown in Figure 6.12.

a) 20% f_y initial PT at 4.5% drift b) 20% f_y initial PT at -4.5% drift



c) Moment at interface versus drift for beam to column testing with armouring

Results from Beam to Column testing with steel armour

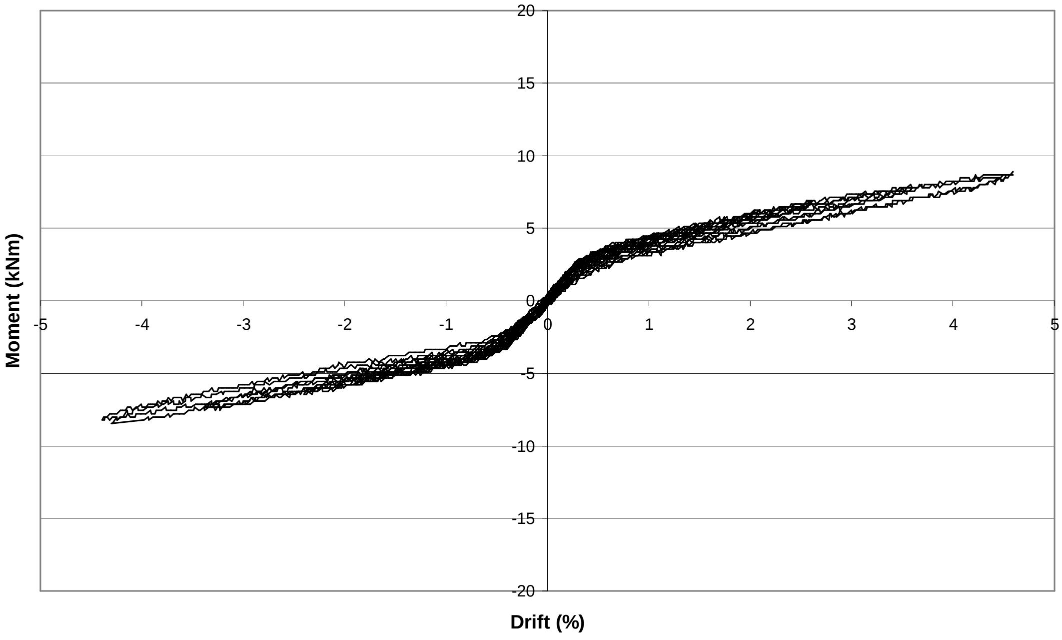


d) 60% f_y initial PT at 4.5% drift e) 60% f_y initial PT at -4.5% drift

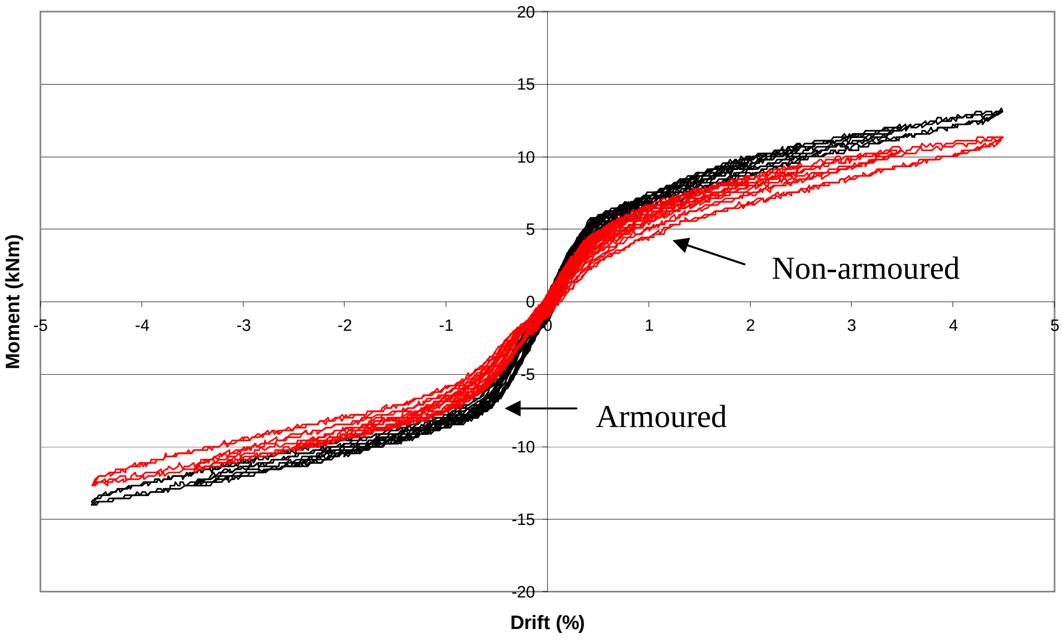
Figure 6.12: Results from Beam to Column testing with steel armour

As shown above the characteristic non-linear elastic behaviour of the post-tensioned only connection was evident in all three tests. The non-linear point was due to a sudden change in the neutral axis depth when the gap first opens at the interface. No permanent damage was observed after any of the testing cycles. The increased in initial post tensioning force increases both the ‘yield’ moment and the ultimate moment. It was also seen that an increase in tendon initial tendon force increased the drift level at which the ‘yield’ occurred, the slope of the pre-yield does not change. This is because the deformation before this point is predominantly the elastic deformation of the column section which does not change.

The second series of three tests used the same initial post tensioning level with the removal of the steel plate. It was expected that this will significantly decrease the moment capacity of the connection as the perpendicular to grain behaviour of the LVL dominates the systems performance. Comparisons made between testing with 40% and 60% initial post tensioning force with and without armouring (Figure 6.13) shows that this is the case.

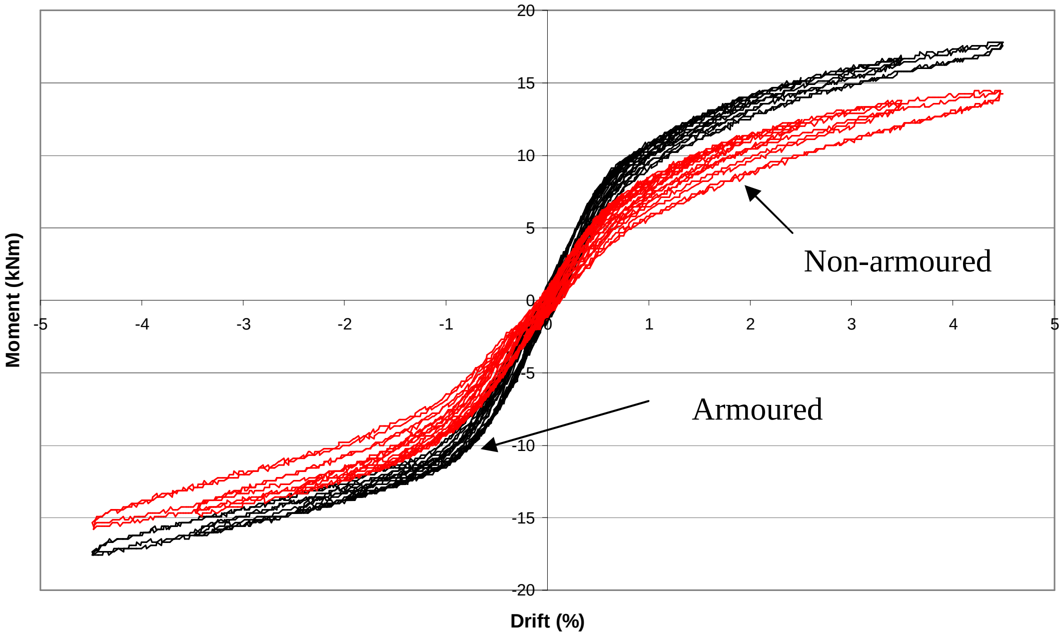


a) Moment at interface versus drift of non-armoured column with 20% initial PT



b) Moment at interface versus drift of armoured and non-armoured column with 40% initial PT

Results from Beam to Column testing with and without steel armour



c) Moment at interface versus drift of armoured and non-armoured column with 60% initial PT

Figure 6.13: Results from Beam to Column testing with and without steel armour

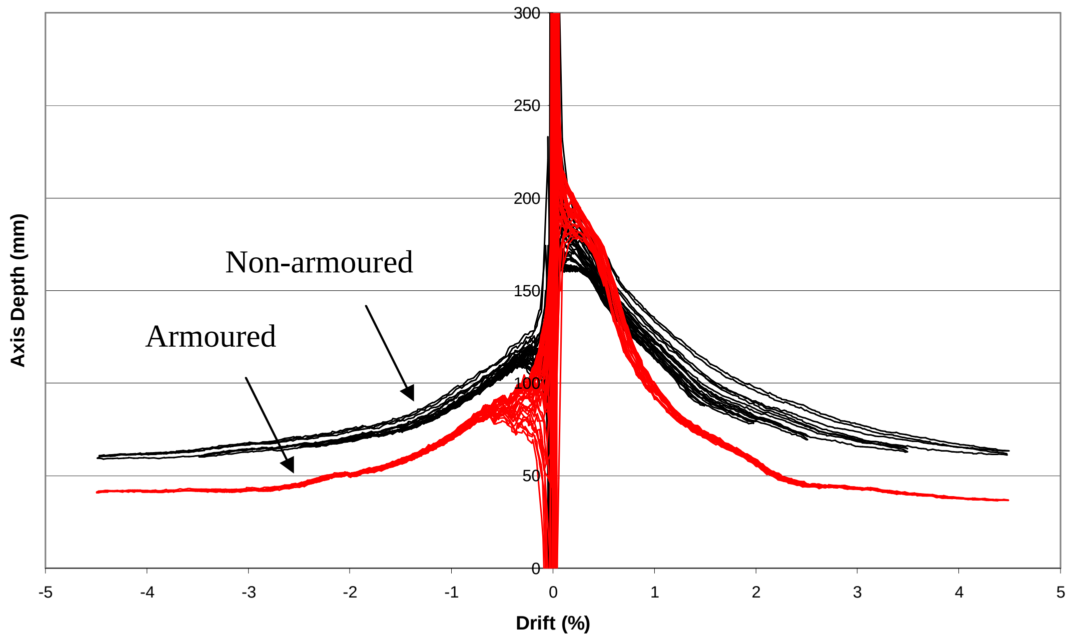
These tests also displayed the characteristic behaviour of the post-tensioned only beam to column connection. It can be seen that at larger levels of drift and at higher levels of post tensioning small amounts of hysteretic behaviour are observed. This corresponds to damage to the column face with some crushing of the fibres stressed perpendicular to the grain.

Figure 6.13b and 6.13c also shows the results from the testing performed on the armoured interface at the same level of post tensioning. Comparing the two the reduced moment capacity of the non-armoured interface is evident. It can also be seen that the non-armoured solution displays slight amounts of hysteretic behaviour. This is due to the small amount of in-elastic damage in the column face. (Figure 6.14)

Figure 6.14: Interface inelastic damage

In order to better understand the effect of the armouring the neutral axis depth (Figure 6.15) and tendon force (Figure 6.16) are plotted against the drift of the assembly and compared for two values of post tensioning, 40% and 60% initial PT (with 40% and 60% representing the initial force in the post-tensioned tendon as a percentage of the yield strength).

40% Initial PT



60% Initial PT

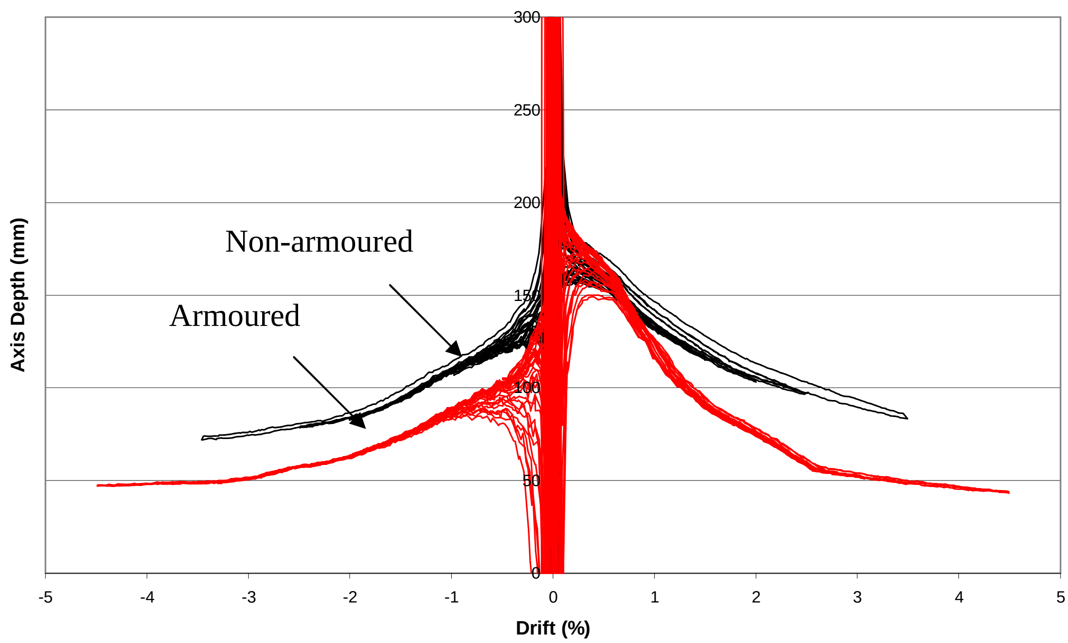


Figure 6.15: Comparison of neutral axis depth vs drift

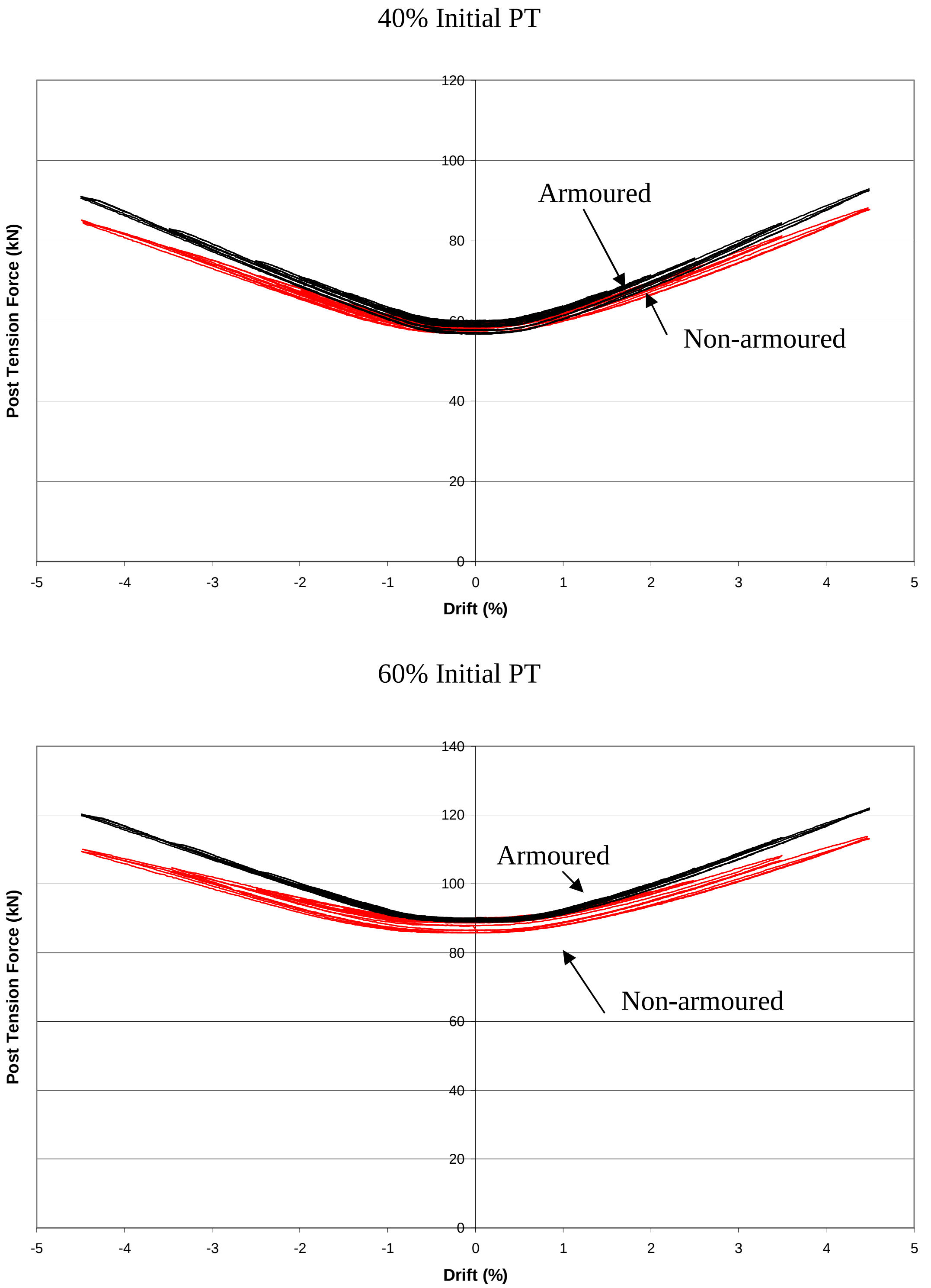


Figure 6.16: Comparison of post tension force vs drift

The effect of the armouring on the neutral axis depth can be clearly seen in both Figure 6.15 and 6.16. The addition of the armour reduces the effect of the softer perpendicular to grain E modulus. This has the effect of increasing the effective modulus at the interface decreasing the compression block and increasing the lever arm of the post tensioning. As the compression block is smaller the gap size at the location of the post tensioning strand increases. This leads to an increased elongation in the tendon further increasing the moment capacity of the connection after the non-linear point. Table 6.3 shows the ‘yield’ and ultimate moments (at 4.5% drift) of the six tests.

Table 6.3: Yield drift and moment, maximum moment for beam to column test

	With Armour			Without Armour		
PT initial	20%	40%	60%	20%	40%	60%
θ_y	0.4	0.5	0.7	0.3	0.4	0.6
M_y (kNm)	3.7	5.3	8.7	2.4	4.0	6.2
M_m (kNm)	10.6	13.3	17.3	8.7	11.1	14.5

Table 6.3 further shows the effect of the armouring. The yield rotation appears to increase slightly when the steel plate is added however, the values are similar and it is not certain whether this trend is characteristic of the system. However, as mentioned in Section 6.3 the ratio between the connection moment capacity and the moment capacity of the beam and column members was not optimised in the design of this subassembly specimen, and it is likely that as the capacity of the connection is optimised (i.e. the ratio between connection and beam strength is decreased) this trend will change. The addition of the steel armour increases the yield moment by approximately 30% and the ultimate moment (at the same drift level) by approximately 17%.

For the design of the six storey building detailed in Chapter 4 the design procedure presented in Newcombe et al. 2008 was used. Figures 6.17, 6.18, 6.19 and 6.20 show the comparison of the moment rotation, neutral axis and tendon force results with the results of this design procedure for the armoured and non-armoured column under different initial post tensioning levels. These predictions use the characteristic values stated in Section 4.2.3 and have not been calibrated.

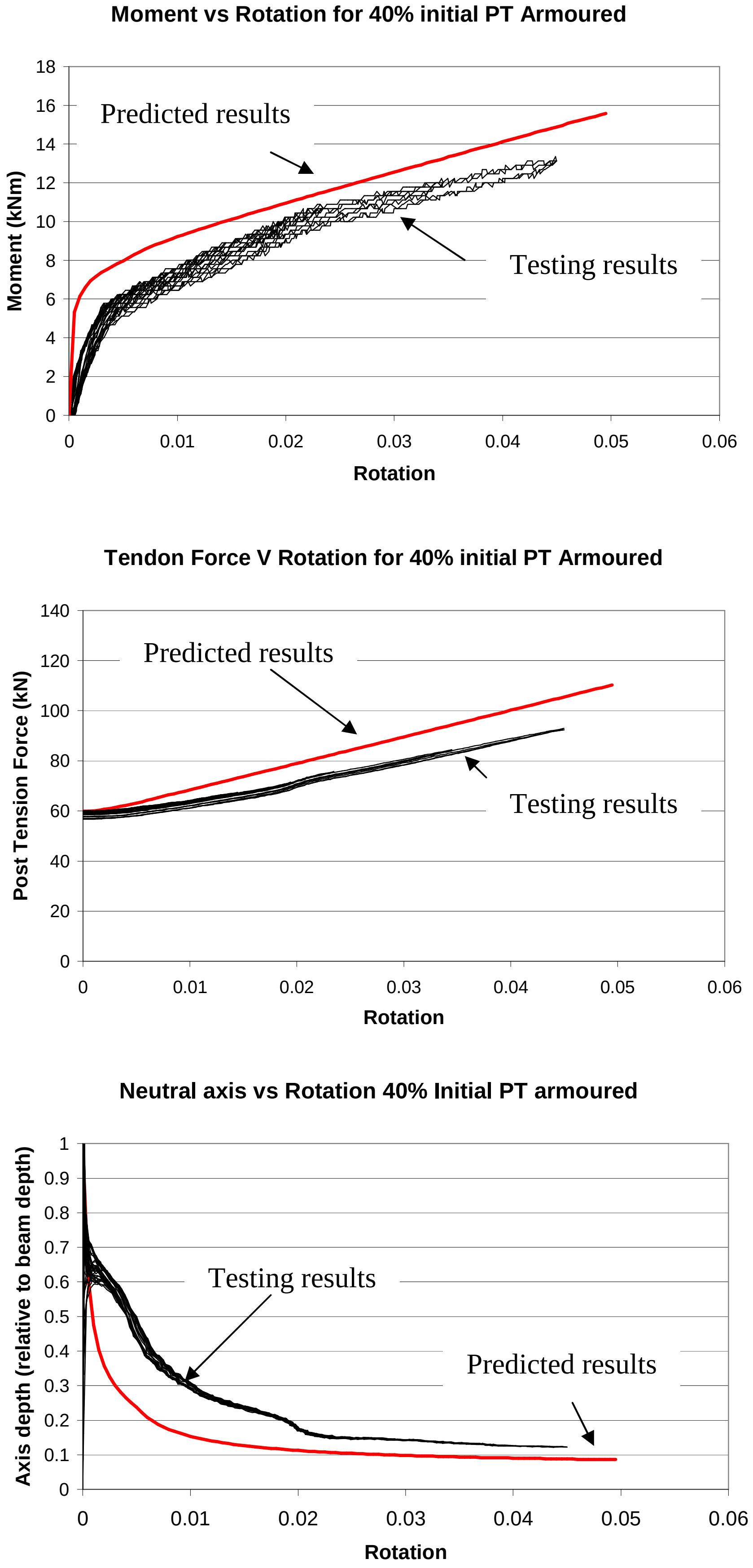
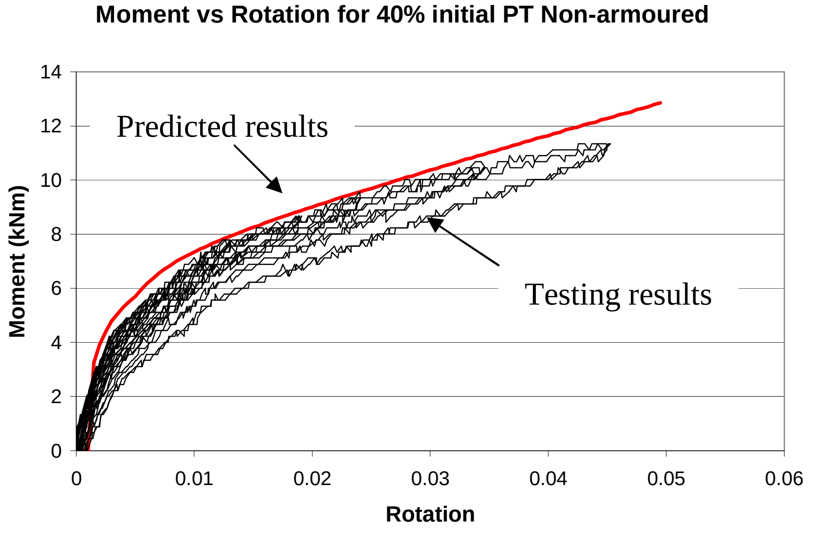
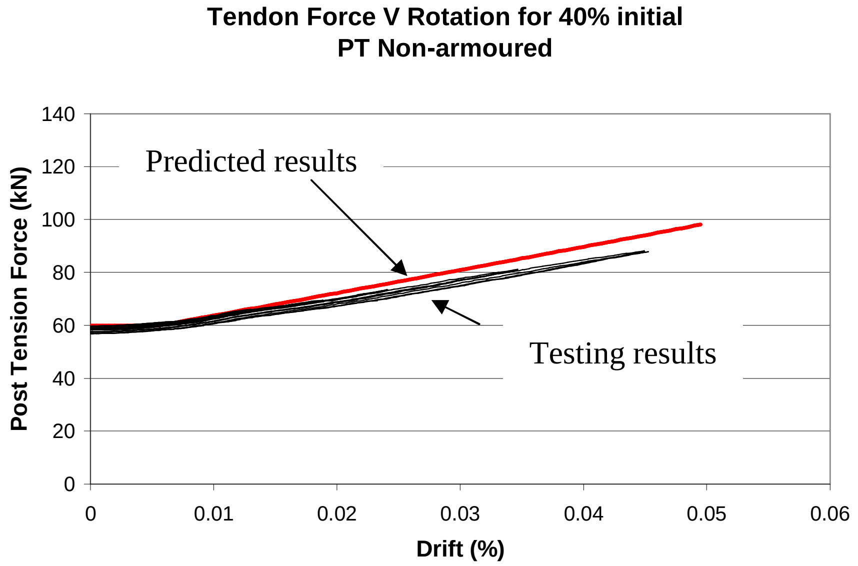


Figure 6.17: Analytical – experimental comparison, armoured 40% PT





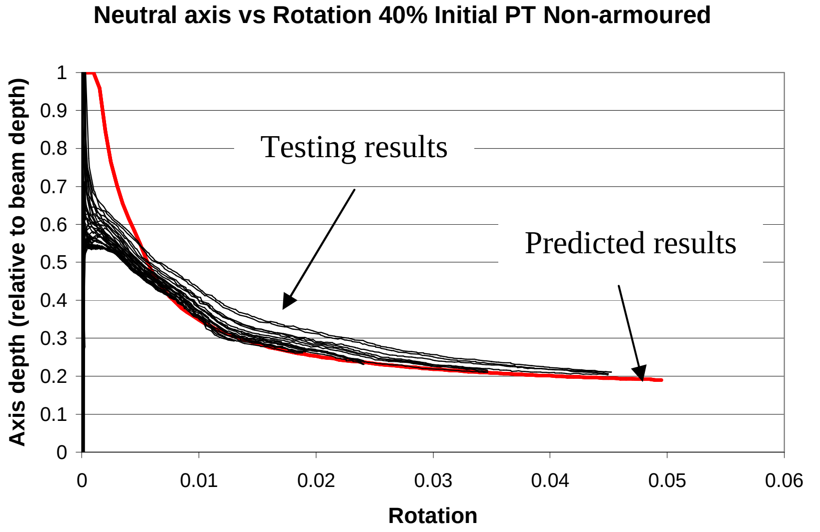
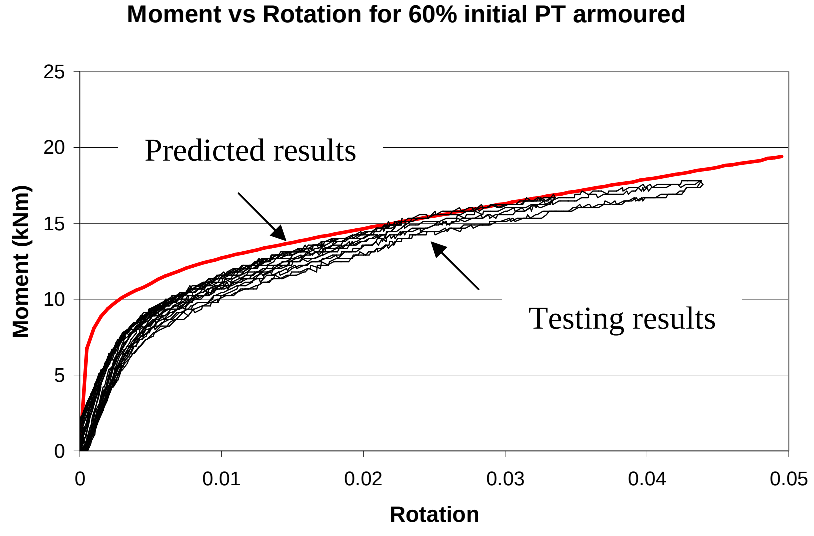
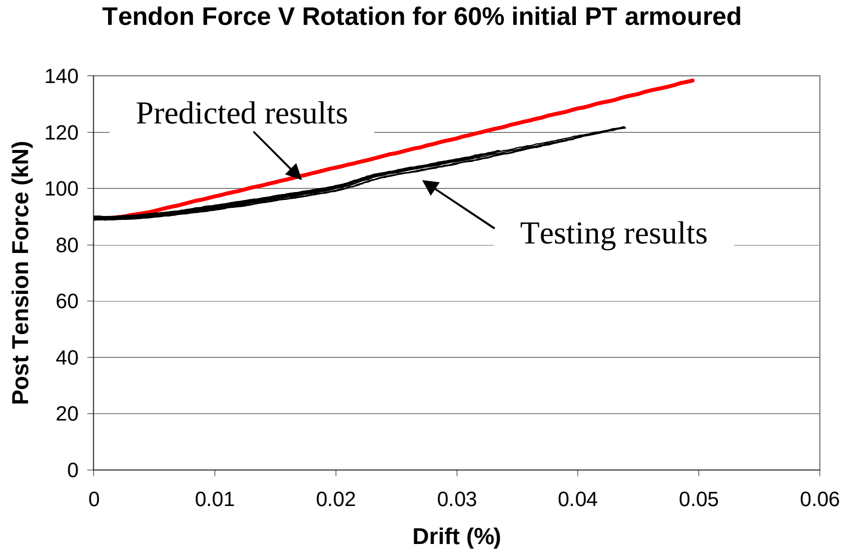


Figure 6.18: Analytical – experimental comparison, non-armoured 40%PT





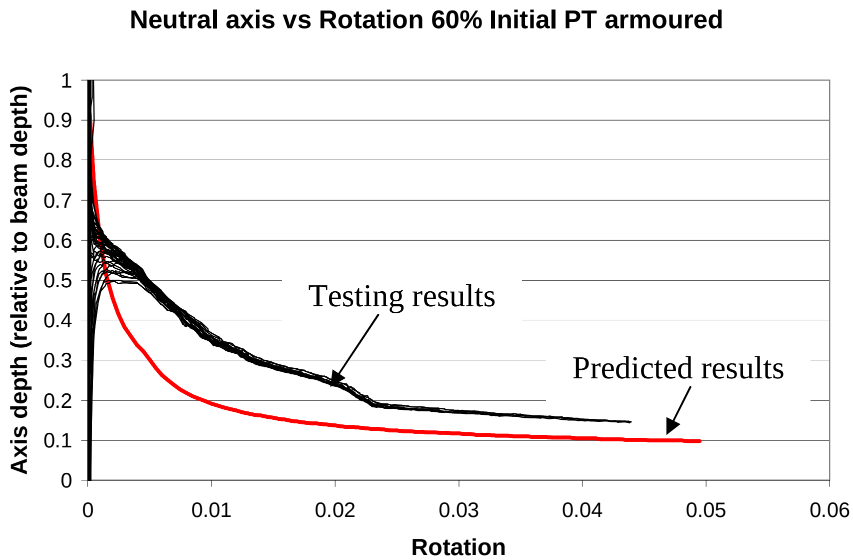
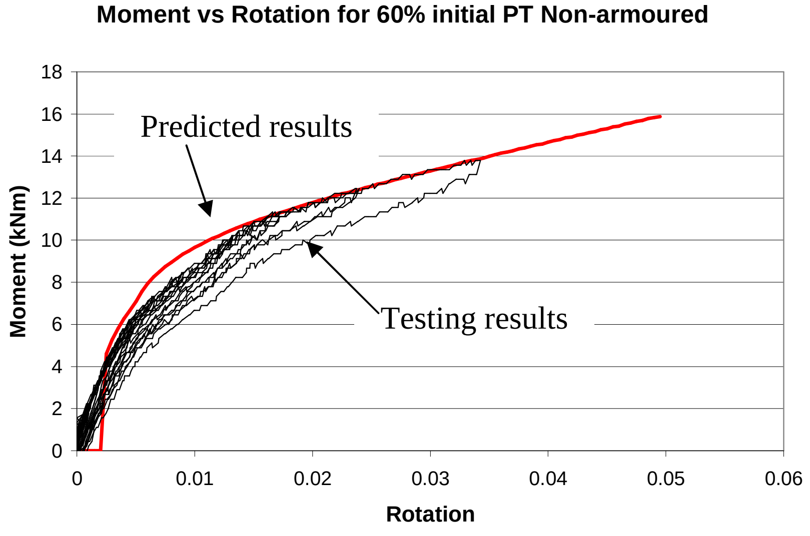
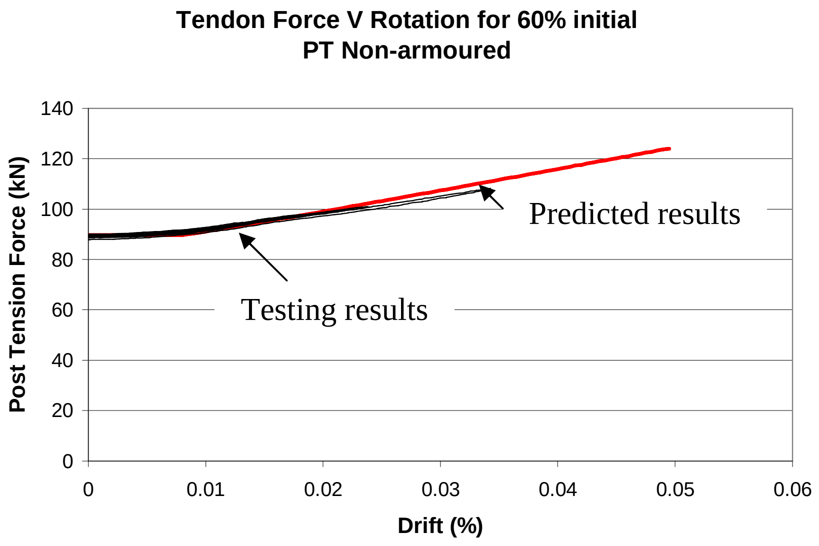


Figure 6.19: Analytical – experimental comparison, armoured 60% PT





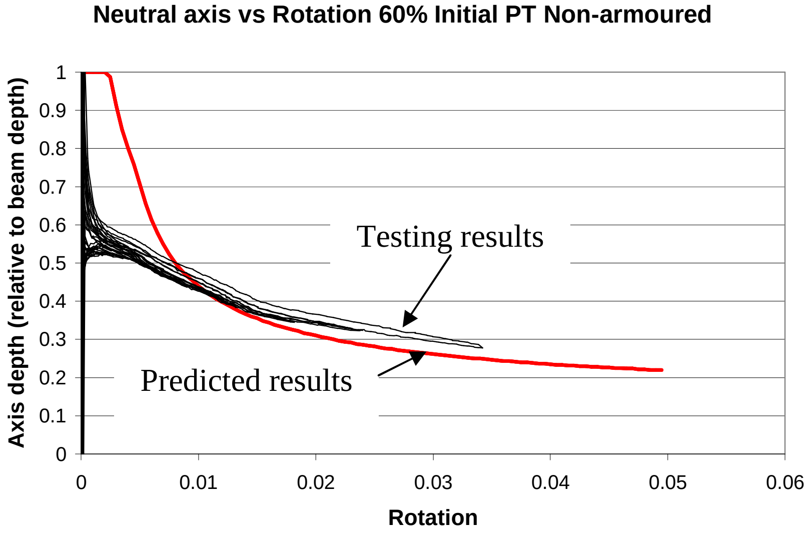


Figure 6.20: Analytical – experimental comparison, non-armoured 60%PT

Figures 6.17, 6.18, 6.19 and 6.20 clearly show that the analytical procedure proposed in Newcombe et al. 2008 adequately predicts the moment rotation response of a timber post-tensioned connection. For higher levels of drift the equations can be seen to be very accurate. From the testing it can bee seen that the procedure is more suited to the prediction of the timber to timber connection, this is due to the calibration of this formula being based on actual beam to column testing. The armoured equation calibration used wall to foundation testing due to the lack of test results. The neutral axis depth is also not exactly captured; this is due to the procedure being calibrated based on the moment rotation response, assuming a triangular stress block. Although this triangular distribution has been observed in testing as shown in Figure 6.21 (Smith 2006b) it is unlikely that the stress pattern remains at higher levels of stress, especially in the case of significant inelastic damage. As the depth of the compression region directly affects the amount of tendon stress, this inaccuracy is also seen in the prediction of tendon force.

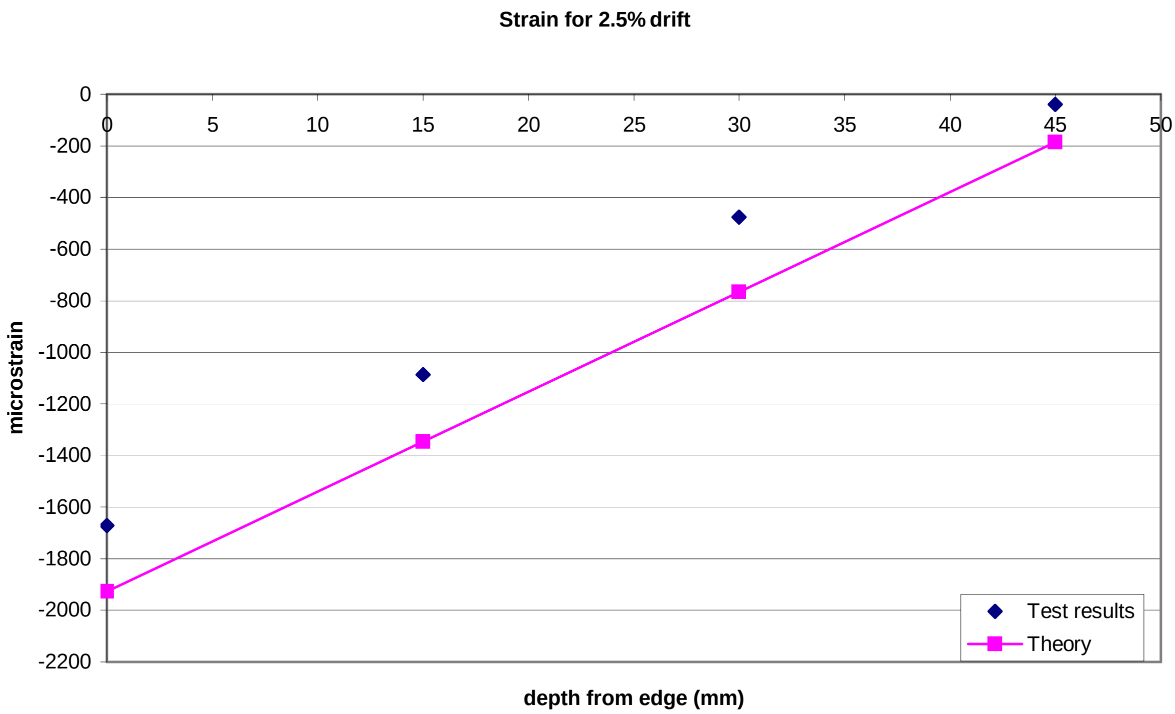


Figure 6.21: Strain at the base of a wall to foundation connection at 2.5% drift (Smith 2006b)

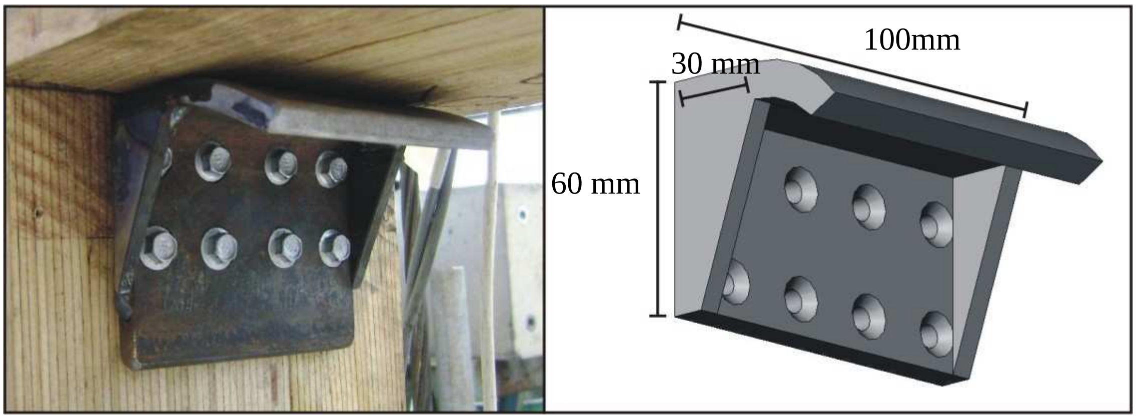
The testing performed in Section 6.3 had two main objectives. The first was to assess the effect of placing armouring on the column face in the connection. The addition of this armouring has the effect of reducing the neutral axis depth by increasing the E modulus of the compression region in the joint. This increase has a significant effect on the response of the joint as the reduction of neutral axis (Figure 6.15) increases the lever arm of the post tensioning. As shown in Table 6.3 this increase in lever arm causes an increase in the both the 'yield' moment and maximum moment (at a certain drift level) of the connection. Further investigation is necessary to asses the effect of armouring if significant inelastic damage occurs in either the beam or column member.

The second objective was to asses the accuracy of the analytical procedure used in the design of the moment connection in the case study building. As discussed above, the predictions adequately capture the moment rotation response of a timber post-tensioned connection and for higher levels of drift the equations can be seen to be very accurate. It is also noted that the method seems more suited to the prediction of a timber-timber interface response. Further refinement will be necessary to capture the effects of inelastic deformation.

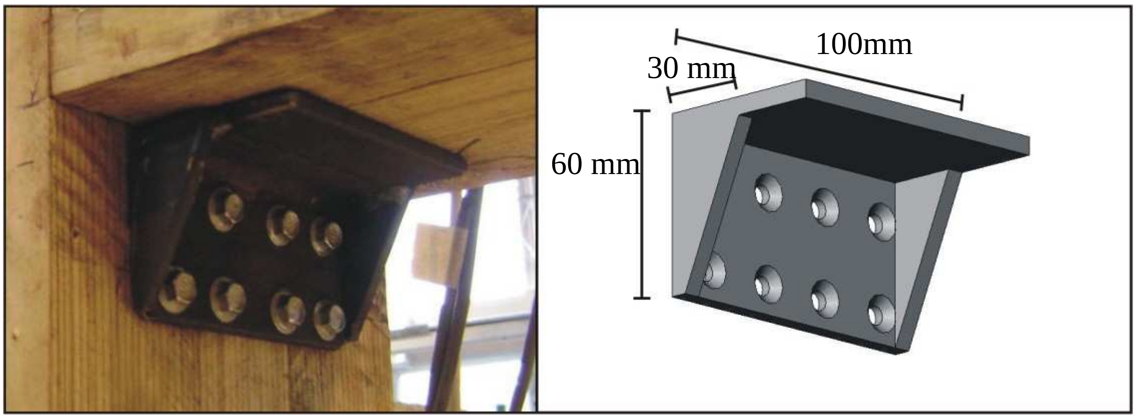
6.4 Testing of Beam Column Interface with Corbels

It has been proposed that for concrete the vertical shear resistance at the beam to column interface can be developed though shear friction, with the post tensioning force provided by the unbonded steel tendons providing a clamping force across the connection (El-Sheikh et. al. 1998, ACI T1.2-03). Despite this fact, many codes still require the attachment of special supports to carry the shear due to the factored gravity loads (NZS 3101: Appendix B). It is important that this connection does not impede the rocking motion of the beam in any way; however, it is still required to carry the factored gravity loading prior to the event. The following section investigates the possible effect of these corbels on the moment response of the beam to column connection.

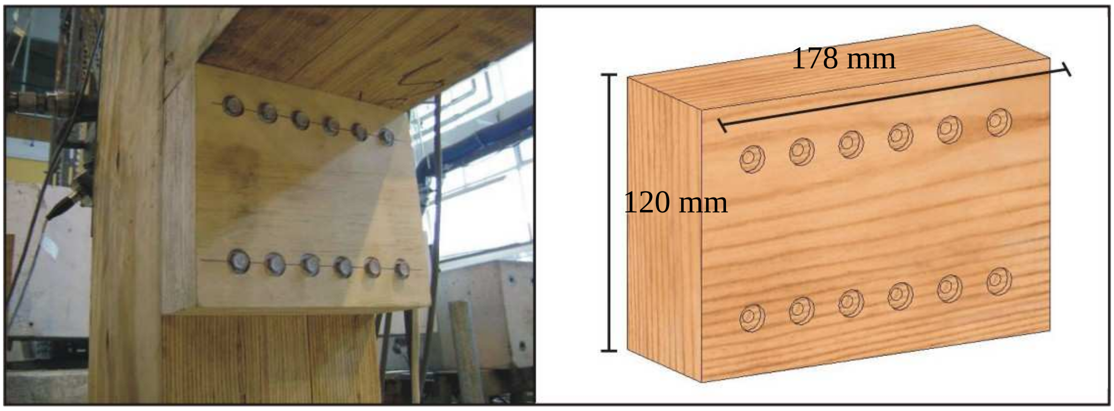
As part of this project three types of corbel are considered, shown in Figure 6.22.



a) Pre-bent corbel



b) Straight corbel



c) Timber corbel

Figure 6.22: Corbel attachments

The corbel attachments were tested using the test set up shown in Figure 6.23. The two steel corbels are stiffened to ensure that 30mm seating length is always maintained. This is the length required to resist the factored gravity loading. A 9kN concrete weight was attached to the top of the beam to provide shear loading equivalent to the seismic gravity loading in the prototype building. Type 17 screws of 175mm length are used to attach the corbel to the face of the beam.

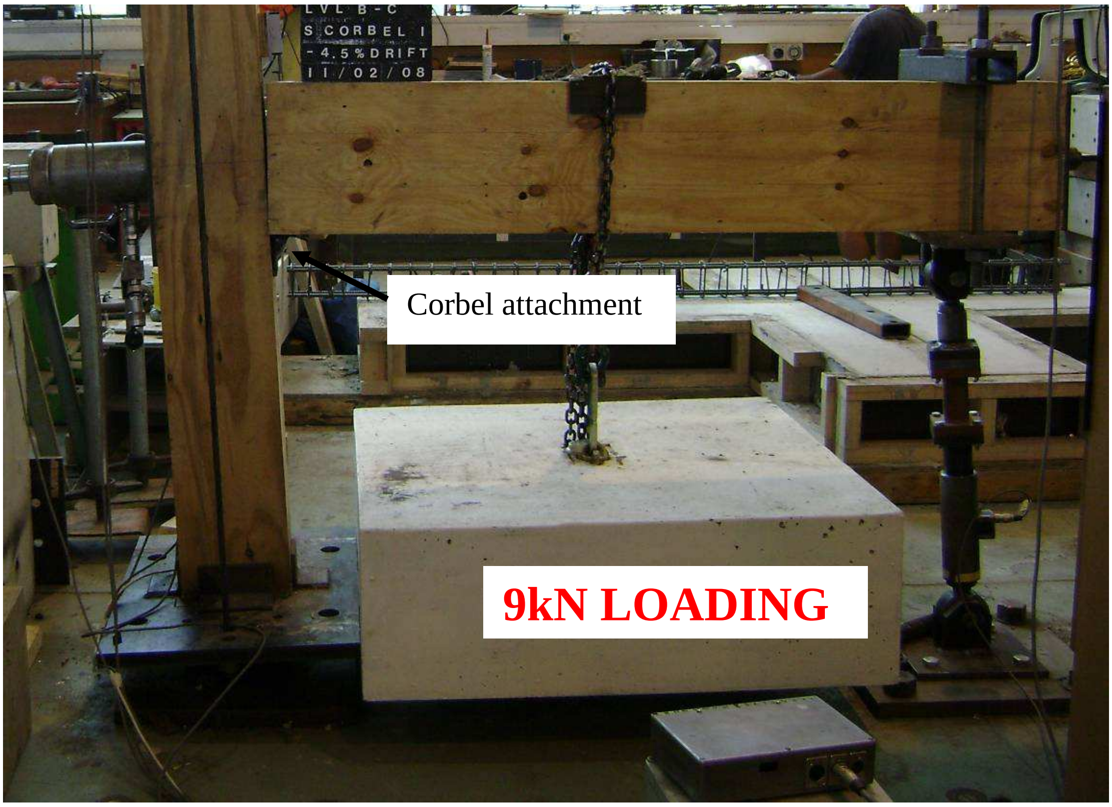


Figure 6.23: Corbel testing set up

The full factored gravity loading is used to calculate the dimensions and fastener requirement for the corbel:

The bearing area of the corbel shall satisfy:

$$A = \frac{1.5V^*}{f'_{c,PERP}}$$

Where:

V^* = Shear loading (from 1.2G + 1.5Q)

$f'_{c,PERP}$ = Compression strength of LVL, perpendicular to the grain

The design of the straight steel corbel was altered from that of the other two corbels. As stated above it is important that the corbel does not impede the rocking motion of the beam on the column. In addition to this it is also important that the corbel does not create significant uplift of the beam relative to the column face (Figure 6.24) leading to unnecessary damage of the flooring. For this reason the first steel corbel was bent

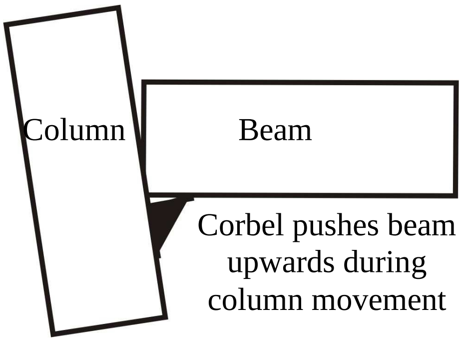


Figure 6.24: Corbel movement during beam rocking

with the intention that the beam will roll over the face of the corbel minimising this uplift effect.

The second steel corbel was designed to yield under this rolling effect. The reduced seismic load was applied as a point loading at the end of the corbel to ensure that the corbel bent. The following design procedure shall be satisfied (Figure 6.25):

$$t = \sqrt{\frac{6V_s d}{1.3 f_y b}}$$

Where:

t = Thickness of the corbel

V_s = Reduced shear loading (G + 0.3Q)

d = Length of the corbel

b = Breadth of corbel

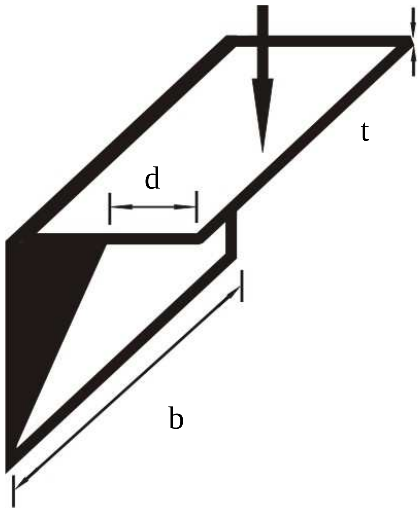
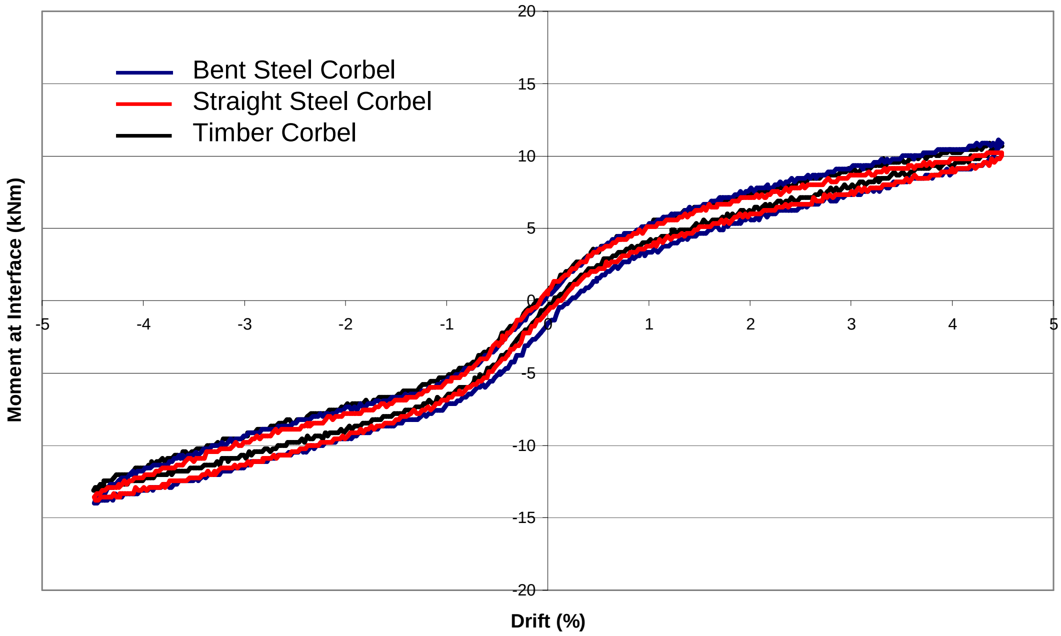


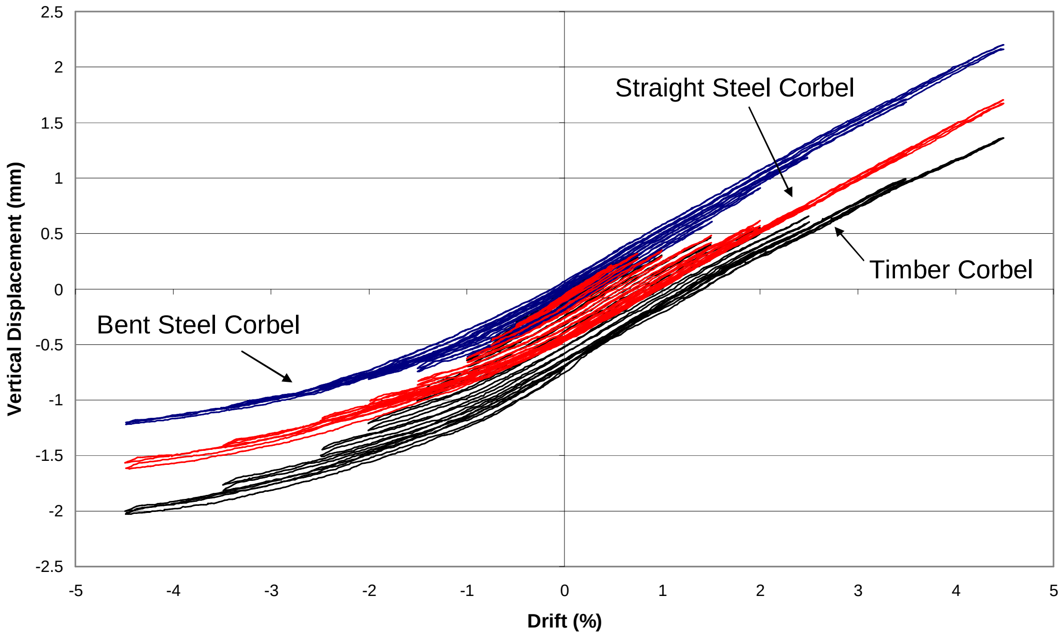
Figure 6.25: Corbel dimensions

The number of screws required for the timber corbel was increased due to it being a timber to timber connection (NZS 3603).

A potentiometer was placed to recorded the vertical movement of the beam, Figure 6.26 shows both this record and the moment rotation results for the three corbel attachments.



a) Moment versus drift for beam to column connection with corbels attached



b) Vertical displacement of beam during testing

Figure 6.26: Results from corbel testing

Figure 6.26a shows that the attachment of the different types of corbel does not a have significant effect on the moment response of the system. For this reason other considerations can be made in the selection of the appropriate type of corbel to be selected. Figure 6.26b shows that there is some movement in the corbel during the cyclic loading as the gravity loading induces slight vertical movement of the corbel.

It has been observed during the cyclic testing of the wall to foundation (Smith 2006b) and column to foundation testing (Pasticier 2006) that although slipping does not occur during loading due to the friction capacity at the interface, a walking movement can occur causing displacement. This is occurring as the corbel is pushed downward by the gravity loading. Further to this it is apparent that the downwards movement of timber corbel is greater than that of the two steel corbels due to the timber attachment being softer than that of the steel connection and minor crushing occur in the timber.

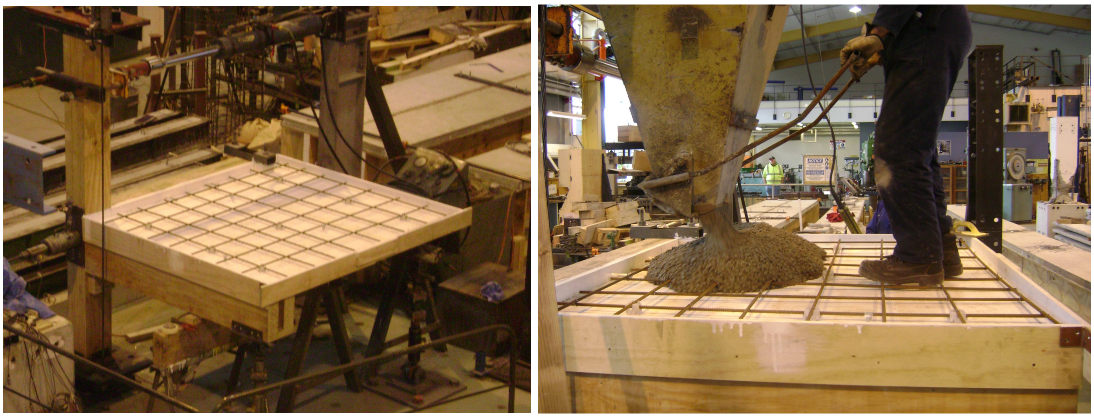
It is important to note that the loading pattern used for the testing is not a good representation of the likely loading pattern arising from a major seismic event. The aim of this testing was to investigate the effect that the corbel attachment had on the moment response of the system. Further to this fact, the attachment of a corbel assists greatly in the construction of the system allowing the beams to be attached without the immediate need for post tensioning. Therefore through adequate design it is possible this attachment can both satisfy the code requirements and assist with the rapidity of construction.

6.5 Testing of Full Flooring Unit

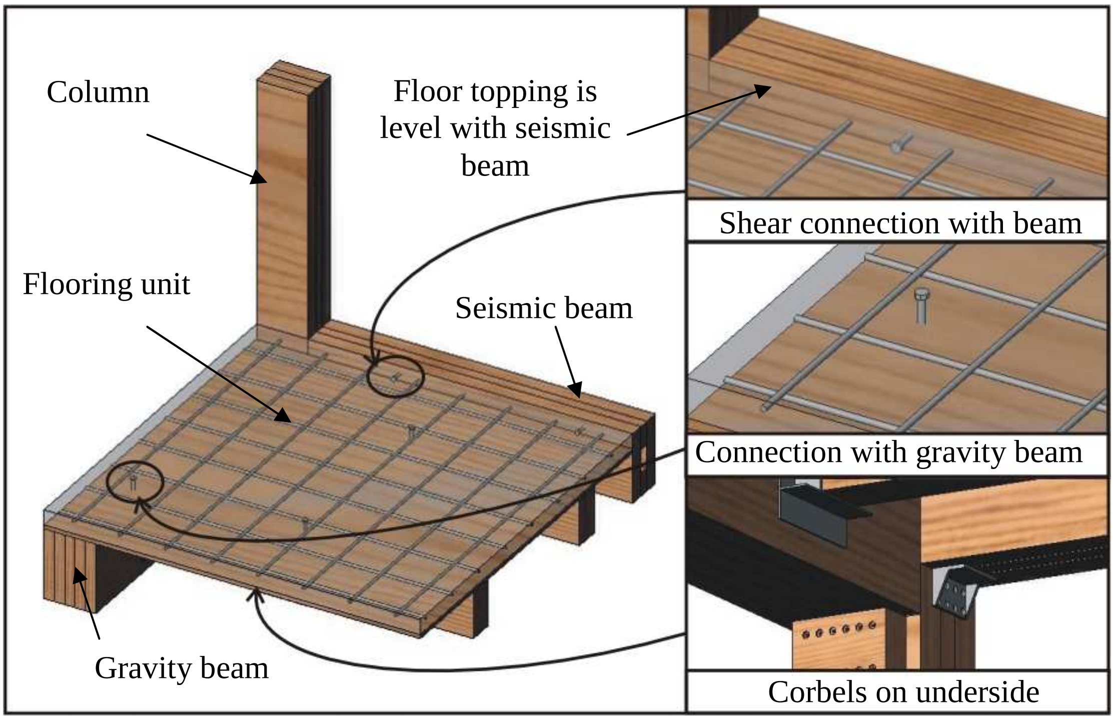
Although experimental testing has been performed to investigate the response of the moment rotation connection and the floor to seismic beam connection, it is the performance of the combination of these that will define the buildings response. To date, testing has not included a floor slab.

6.5.1 Test Setup

In order to investigate the buildings global response a floor unit was added to the beam to column connection. This new testing setup is displayed in Figure 6.27.



a) Floor before and during pouring of floor topping



b) Details of beam to column subassembly with floor

Figure 6.27: Beam to column subassembly with floor

As shown, the flooring is supported on joist which are connected to a gravity beam though the use of joist hangers (Figure 6.28a). The beam is then seated on a timber corbel which is attached to the seismic column (Figure 6.28b) this is a gravity connection only and no lateral attachment to the column was placed. The floor unit is connected to the frame by the coach screw connection tested in Section 6.2 (Figure 6.28c) which was tied into the mesh consisting of HD10 bars at 200 centres.

Figure 6.28: a) Joists hanger b) Timber corbel c) Lateral coach screw connection

As this is a corner unit it was suspected that the gravity beam may rock unrestrained on the timber corbel (Figure 6.29a) due to there not being an attachment from the column into the slab. This rocking would create a considerable moment to occur in the timber corbel (Figure 6.29b). Therefore the screws were checked for this combined loading.

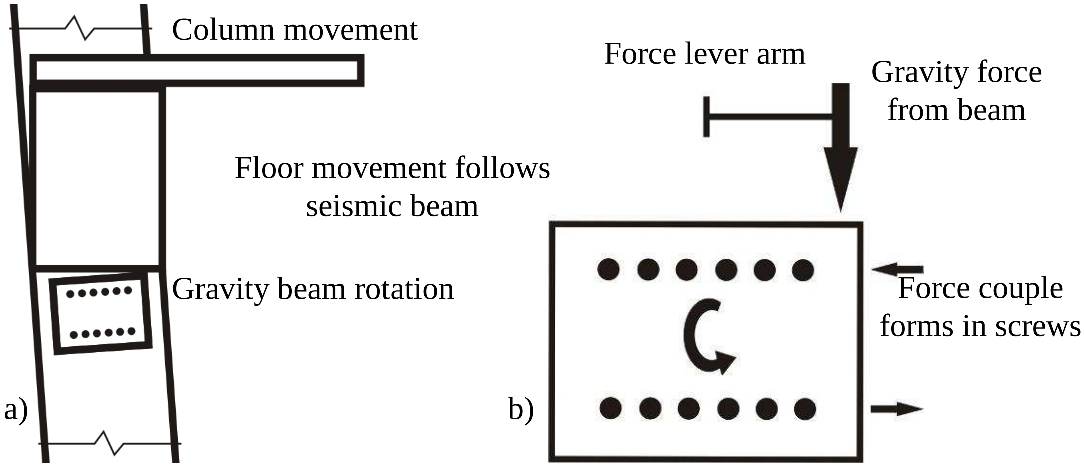


Figure 6.29: a) Movement of gravity beam b) Corbel force couple

6.5.2 Test Results

Figure 6.30 shows the moment drift results of the test. The following paragraphs offer a detailed analysis of the sub-assembly performance under the loading protocol.

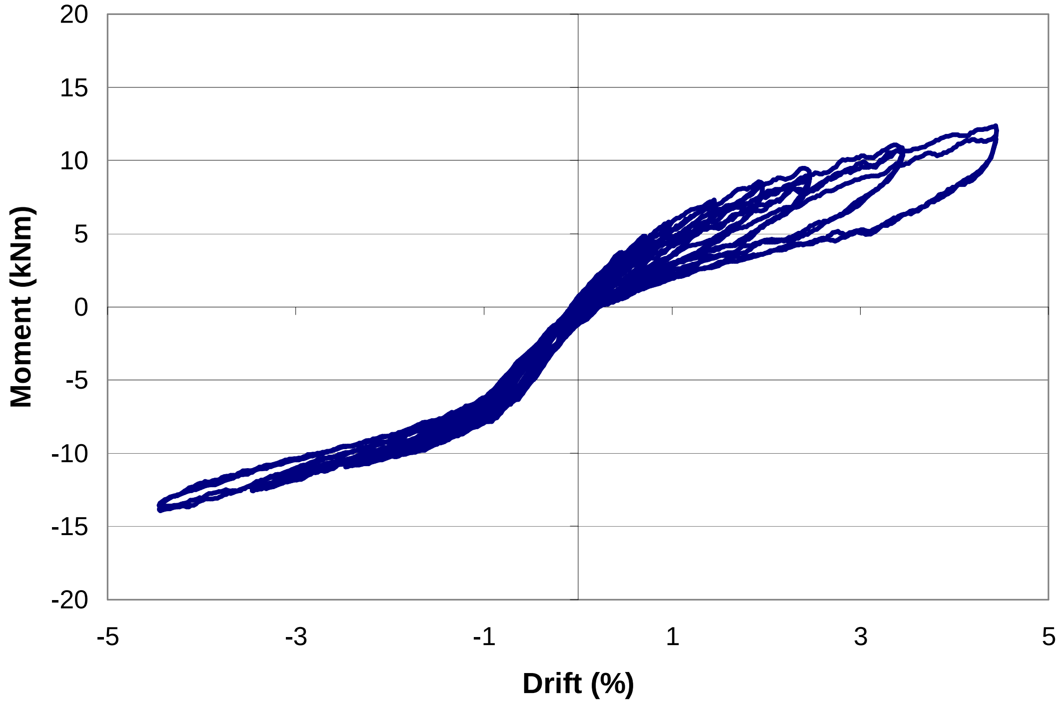


Figure 6.30: Moment verses drift response of sub-assembly with floor

Clear antisymmetric behaviour can be seen in Figure 6.30 and the assembly is providing hysteretic behaviour in the positive direction. Although this hysteretic behaviour is occurring in the system full recentering is still achieved.

6.5.3 Observed Floor Damage

Figure 6.30 shows clearly the antisymmetric behaviour observed during testing, however, no major damage to the flooring unit occurred during testing (Figure 6.31).

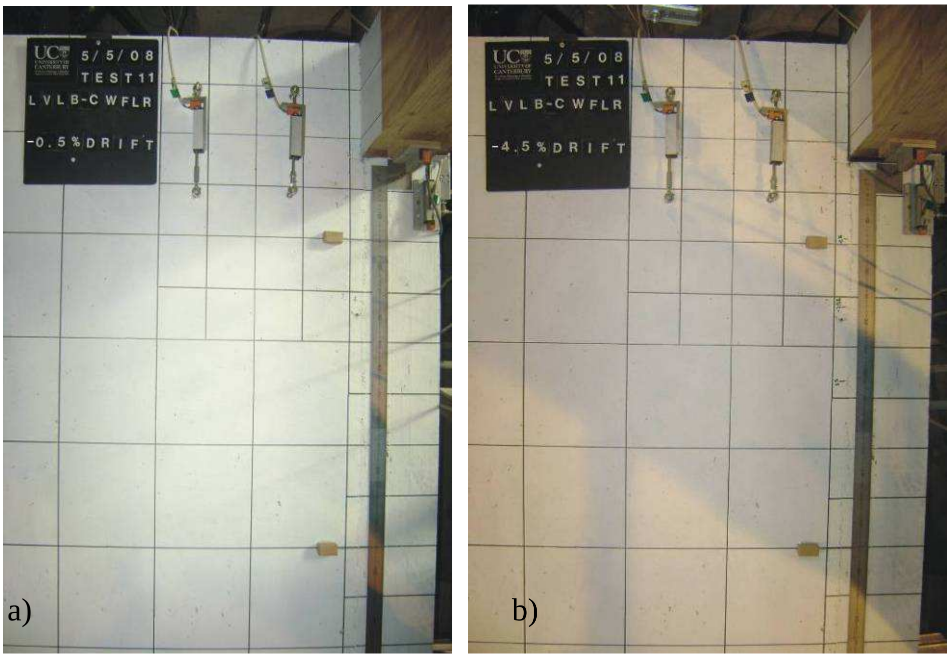


Figure 6.31: Plan view of flooring unit during: a) -0.5% drift b) -4.5% drift

Upon initial movement of the column the floor unit unattached from the column and at large drift levels considerable relative movement between the floor slab and the column were observed (Figure 6.32a). Due to the top gap opening this movement was considerably larger in the negative direction. During larger drift levels it was noted that a single large crack was opening causing vertical displacement between the floor and the post-tensioned beam (Figure 6.32b). It was noted that this crack only appeared during positive drift movement. In order to understand this displacement the geometry of the section must be considered.

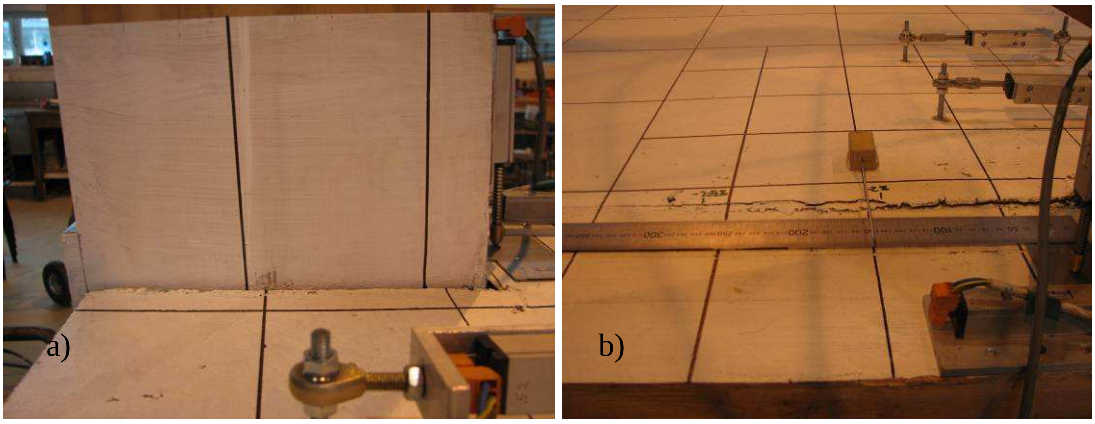


Figure 6.32: a) Relative movement between the floor slab and column -4.5% drift
b) Crack appearing at +4.5% drift

During the increasing level of drift experienced by the specimen it was noted that the gravity beam was rocking on the timber corbel (Figure 6.33a). This deformation was larger in the negative drift direction than it was in the positive (Figure 6.33b).

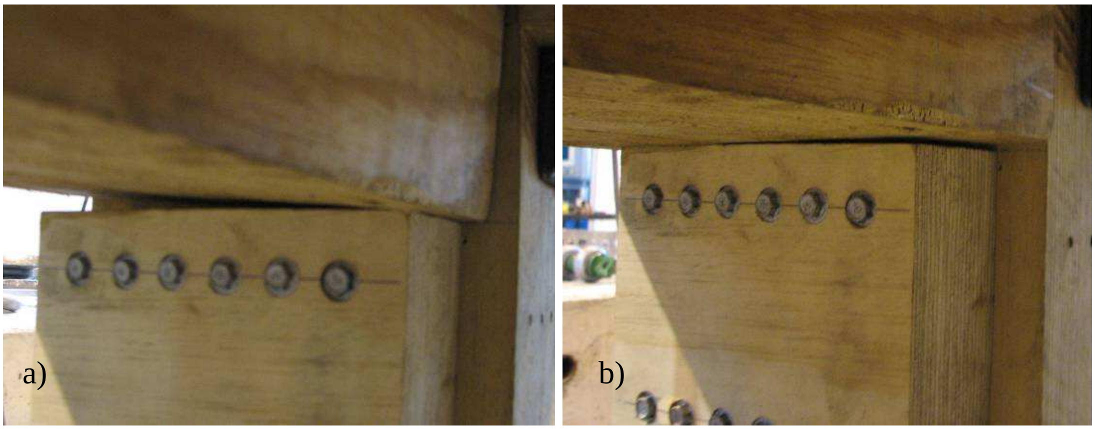


Figure 6.33: Beam rocking on corbel at a) -4.5% and b) +4.5%

These observations lead to an understanding of the antisymmetric response of the system. During negative column drift the beam is being push vertically by the steel corbel attached to the column face and the friction at the beam column interface, in addition to this the gravity beam is being pushed vertically by the timber corbel. This causes the beam to rise off the seating on the far side due to the moment and tension capacity of the slab (Figure 6.34).

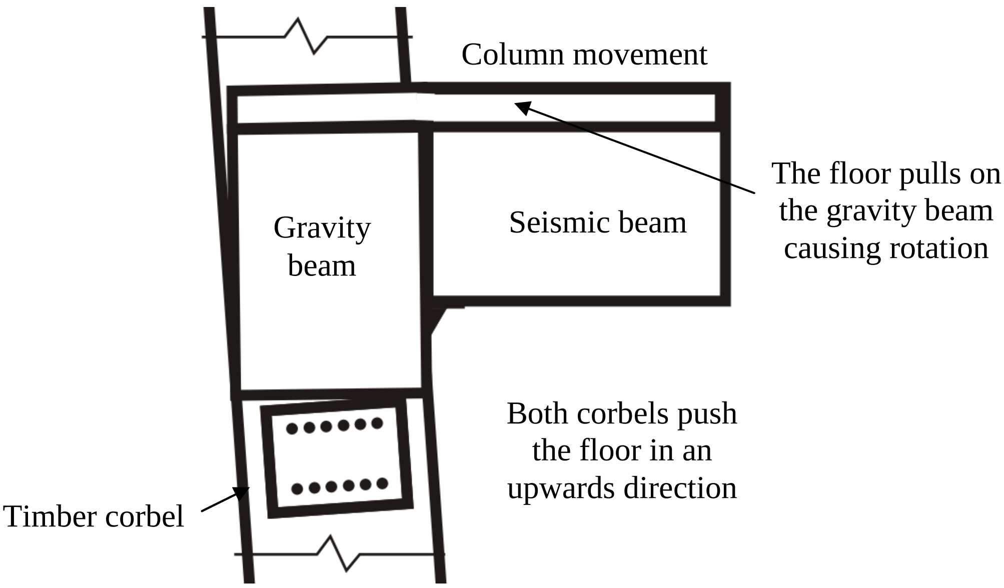


Figure 6.34: Movement of gravity beam during negative drift

In contrast during positive drift the steel corbel will not have an effect on the beam and the friction force will act in the opposing direction, resisting the movement of the gravity beam. This causes the tearing in the flooring unit along the face of the seismic beam. This damage in the flooring is in turn leading to the hysteretic behaviour through inelastic deformation in the floor (Figure 6.35).

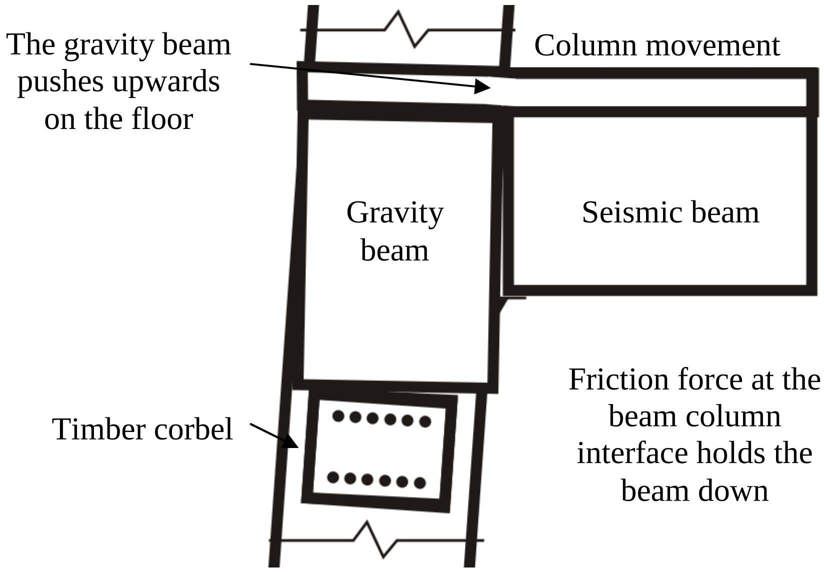


Figure 6.35: Movement of gravity beam during positive drift

The effect of this can be further seen when comparing the movement of the beam in relation to the column face and comparing it to the vertical movement of the beam without the floor attached shown in Figure 6.36.

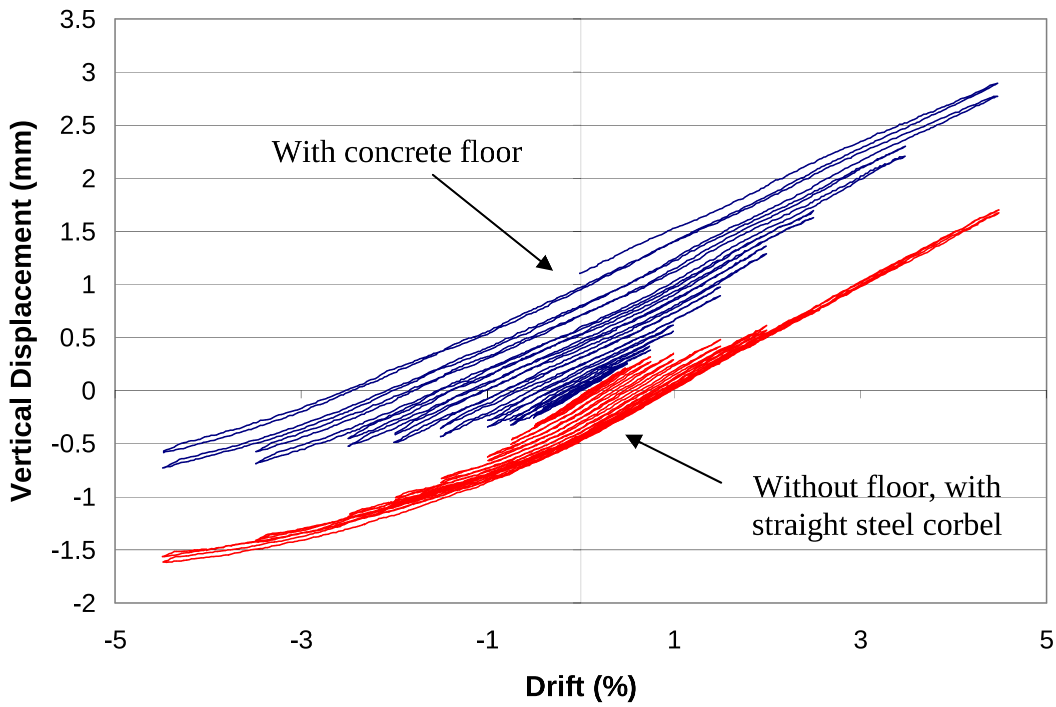


Figure 6.36: Vertical beam movement during cyclic loading

Figure 6.36 clearly shows the upward movement of the beam relative to the face of the column with a final residual displacement of 1.1mm occurring.

6.5.4 Joist Movement During Testing

As mentioned in Section 5.1 a gap should be left between the end of the joist and the face of the gravity beam (shown in Figure 6.37a) to ensure the joist have a minimal effect on the rocking at the beam to column interface. The joist is also not attached to the joist and is simply seated on the hanger. This is to allow for movement of the joist as the gravity beam rotates during lateral movement of the building (Figure 6.37b).

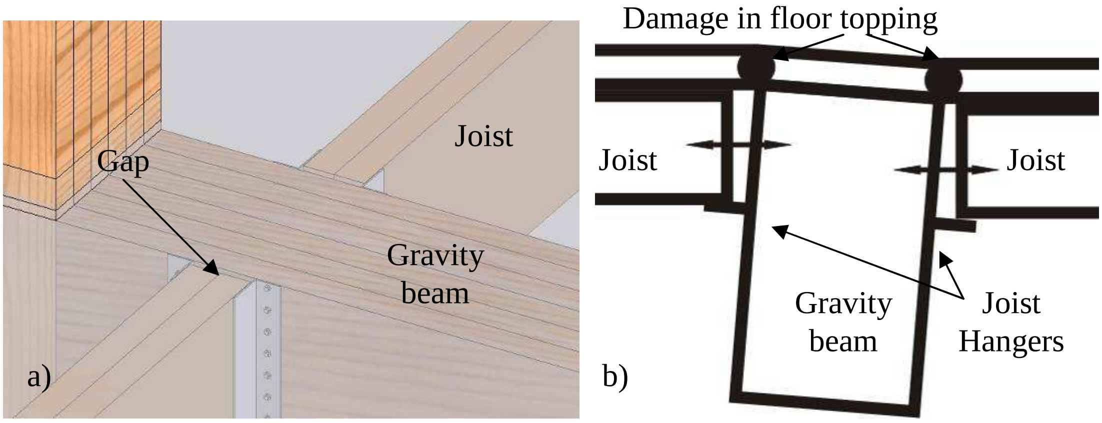


Figure 6.37: a) Joist connection with gravity beam b) Gravity a joist movement during lateral loading

Although Figure 6.31 clearly shows that no damage occurred in the flooring unit this behaviour was clearly observed on the underside of the joist (Figure 6.38). A mark was placed on the underside of the beam at +4.5%, therefore as the photo has been taken at -4.5% the distance between the underside of the joist hanger and the mark is the movement experienced. Clearly the rotation of the joist in the hanger can be seen as the distance between the mark and the hanger face decreases up the joist (Figure 6.38). Although this rotation does occur no damage in the floor is experienced.

Figure 6.38: Joist movement

6.5.5 Effects of Flooring on Moment Response

It is often assumed in seismic design that the floor has little or no effect on the moment response of the lateral resisting system. However, flange effects in beam to column connections can have a significant effect, possibly altering the strength hierarchy of the connection. Figure 6.39 compares the beam to column connection with and without the flooring unit attached.

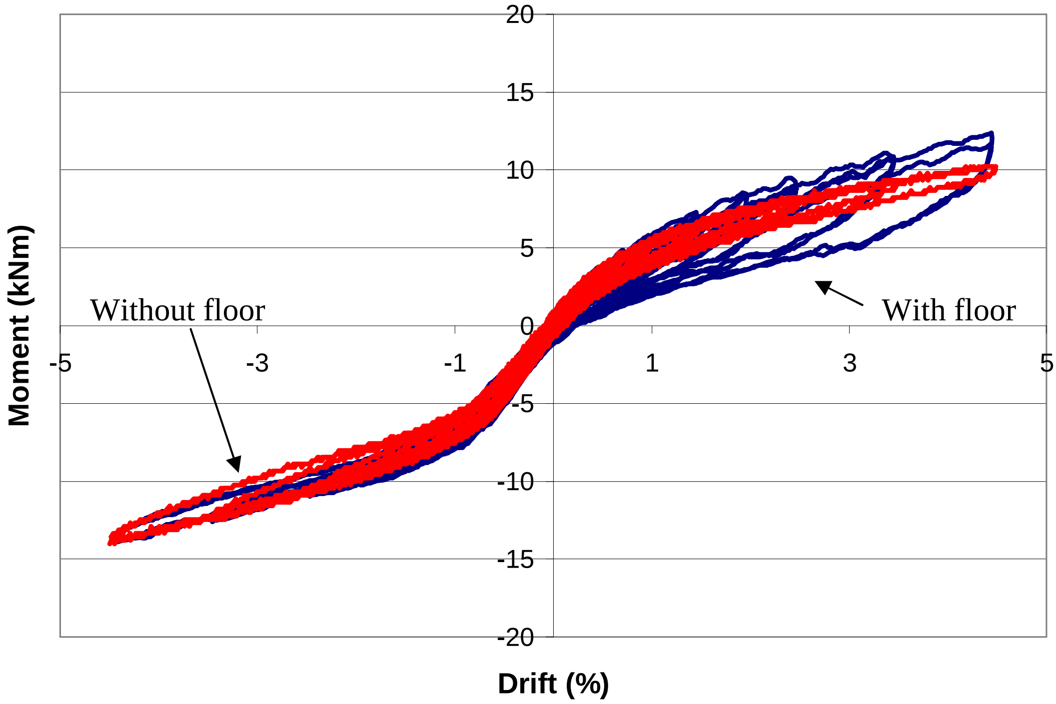


Figure 6.39: Moment rotation response with and without flooring unit

Figure 6.39 above clearly shows the effects of the floor contribution in the positive drift direction adding a small amount of addition moment capacity and hysteretic behaviour. In contrast to this the negative direction show has no effect on the moment response of the system. Finally a comparison was made between the +4.5% drift cycles with and without the addition of the slab (Figure 6.40).

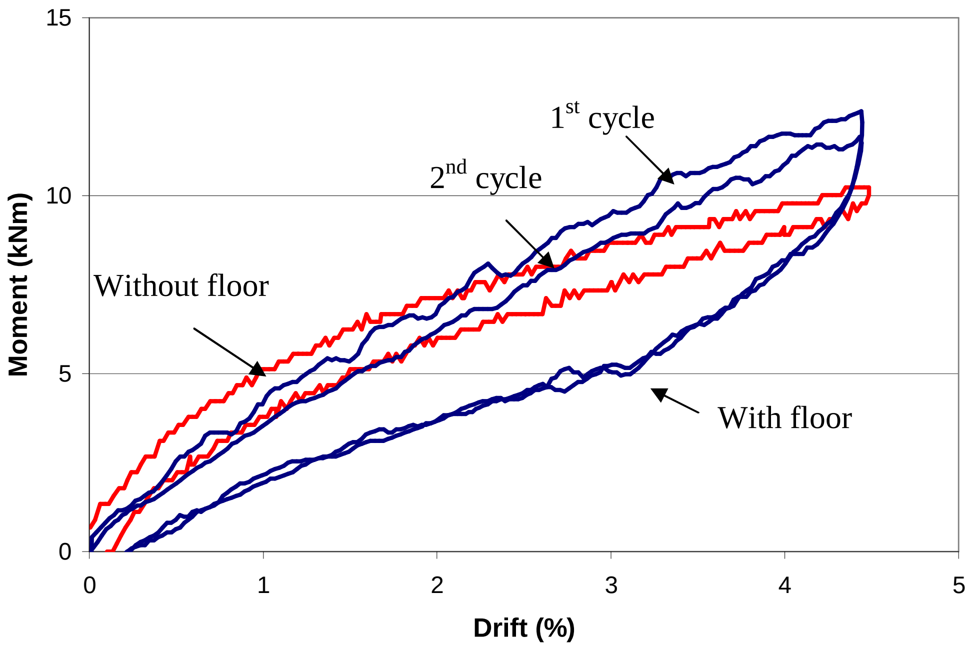


Figure 6.40: Moment verses drift at +4.5% with and without the addition of the floor

Figure 6.40 shows reduced moment response in the second cycle due to the damage in the flooring to seismic beam connection and the hysteric behaviour is clearly seen. It is also apparent that the flooring unit causes a loss of the geometric non-linear point that is characteristic in a ductile post-tensioned connection. It is likely this is due to some load sharing occurring between the seismic beam and the slab, causing a change in the neutral axis depth leading to a softening of the connection. Further research is required to establish the significance of this possible load path.

Overall, it can be seen that the damage of a flooring unit will have a significant effect on the moment response of the beam to column connection. Due to the asymmetric nature of the subassembly an asymmetric moment response was observed. Because the gravity beam was allowed to rock on the corbel tearing occurred at the interface between the flooring unit and the seismic beam, however, the gap opened at maximum drift closed as the column with no residual displacement making repair simple and cost effective.

6.5.6 Effect of an Interior Joint

Although this test had very pleasing results with minimal damage to the flooring unit, it is recognised that this behaviour will not occur for an interior joint (Figure 6.41a). During the rocking of the beam against the column the gap opening must be accommodated by tearing in the flooring unit (Figure 6.41b). The damage in the flooring unit will most likely lead to an increased hysteretic behaviour and a similar loss of the non-linear point observed in the testing with the floor unit performed. It is likely similar damage will occur in an external beam to column joint if the gravity beam is restrained in some way (i.e. tied down to the corbel).

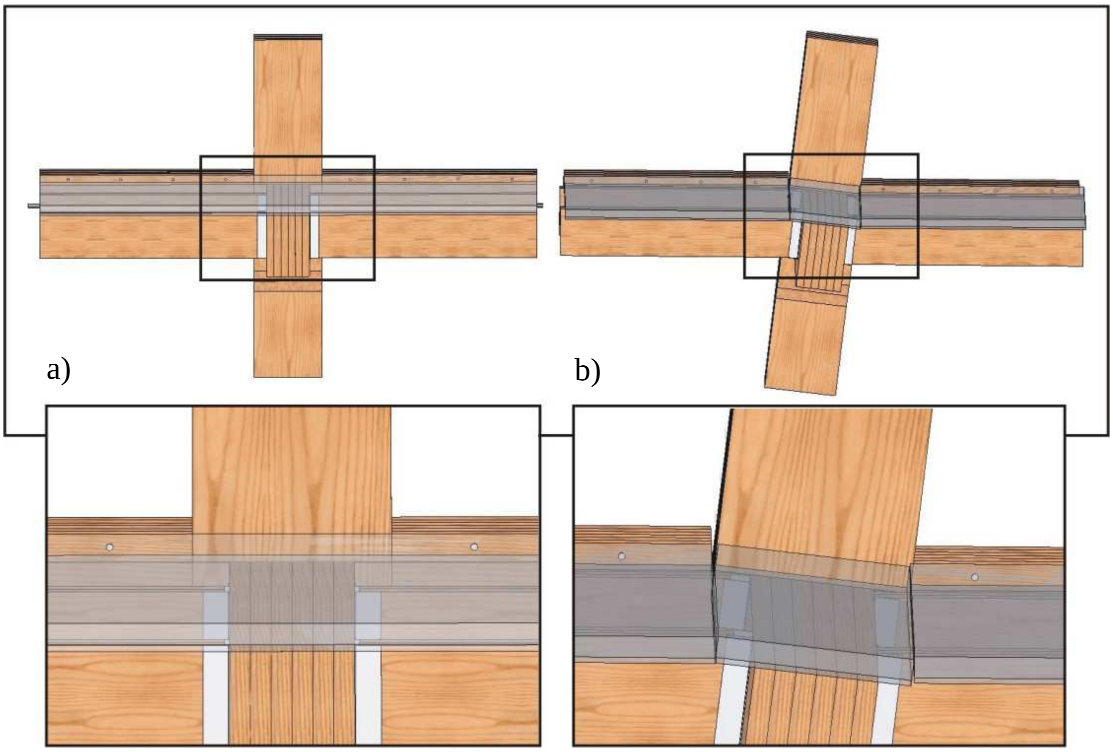


Figure 6.41: a) Interior joint without rocking movement
b) Floor tearing during frame rocking

Clearly further research is required in order to understand and predict the likely behaviour of this interior connection.

6.6 Conclusions Drawn from Subassembly Testing

At the beginning of this section a series of questions was asked regarding the subassembly testing to be performed. Summary answers to these questions are listed below:

- The usage of angled shear keys at the base of a wall or column is preferable to that of the half circular shear keys as it reduces stress concentrations and damage.
- A minimum characteristic strength of 10kN is suggested for the beam to floor diaphragm coach screw connection due to this being the minimum value of

the observed onset of non-linear behaviour, however, larger values than this may occur followed by a sudden slip failure.

- The placement of steel armour at the beam to column interface causes a significant increase in both 'yield' moment and maximum moment (at 4.5% drift) by reducing the neutral axis depth.
- Altering the initial post tensioning value in the column has the effect of increasing 'yield' drift, 'yield' moment and maximum moment (at 4.5% drift).
- The design procedure developed as part of the post-tensioned timber research project at the University of Canterbury (Newcombe et al. 2008) adequately predicts the moment response of a beam to column connection but is more suited to a timber to timber connection. Further research is needed to asses the effect of inelastic behaviour in the joint.
- The placement of corbels on the underside of the beam does not effect the moment response of the beam to column connection
- The placement of a floor unit on the beam to column subassembly caused unsymmetrical hysteretic behaviour to occur (from damage to the floor) due to the unsymmetrical nature of the specimen. Further research is required to understand the effects of the flooring unit on an interior joint.

7 Construction

Constructibility of any new system is seen as being crucial to the feasibility of that specific construction method. This chapter outlines the method and assembly of key components of the proposed post-tensioned timber building. A comparison is also made between the construction time of the timber and the precast concrete case study buildings.

7.1 Construction Method of Timber Building

Well planned construction methodology can dramatically reduce the amount of time taken in the assembly of a structure. It is crucial that the construction method utilises the off-site prefabrication of the timber members as one of the key advantages of the post-tensioned timber system. In order to assist the rapidity of construction the building was separated into three sections (Figure 7.1) enabling workers to perform tasks on separate sections without conflict. The proposed construction procedure is detailed in the following paragraphs.

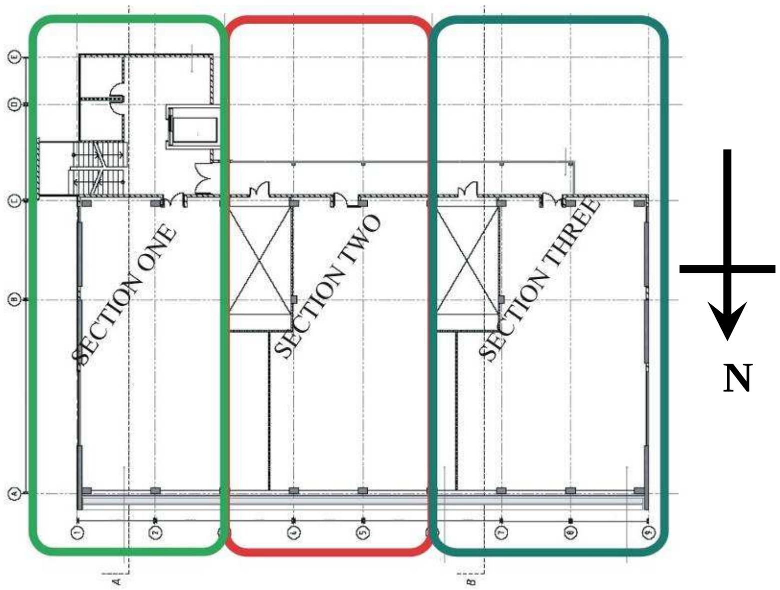


Figure 7.1: Building construction sections

7.1.1 Platform and Balloon Construction

Two main types of construction method exist for the erection of light timber buildings. These methods can also be used for the erection of a post and beam structure (Buchanan 2007) such as the post-tensioned LVL system. The first of these methods is *platform* construction, shown in Figure 7.2a, in which the building in constructed on a floor by floor bases. This mean the column and wall segments will be a single storey high. This method has the advantage of providing a consistent working platform for the floor below. In the construction of light timber frame buildings this method is not recommended for buildings above four storeys as crushing due to perpendicular to grain loading will become a problem. The second method is that of *balloon* construction. Shown in Figure 7.3b, the columns and walls are continuous over several storeys and beams and flooring is then attached up the height. This method can save construction time due to less members being assembled on site. The prefabrication of members, added to the lightness of timber, means that the balloon construction method is preferred for post-tensioned timber construction.

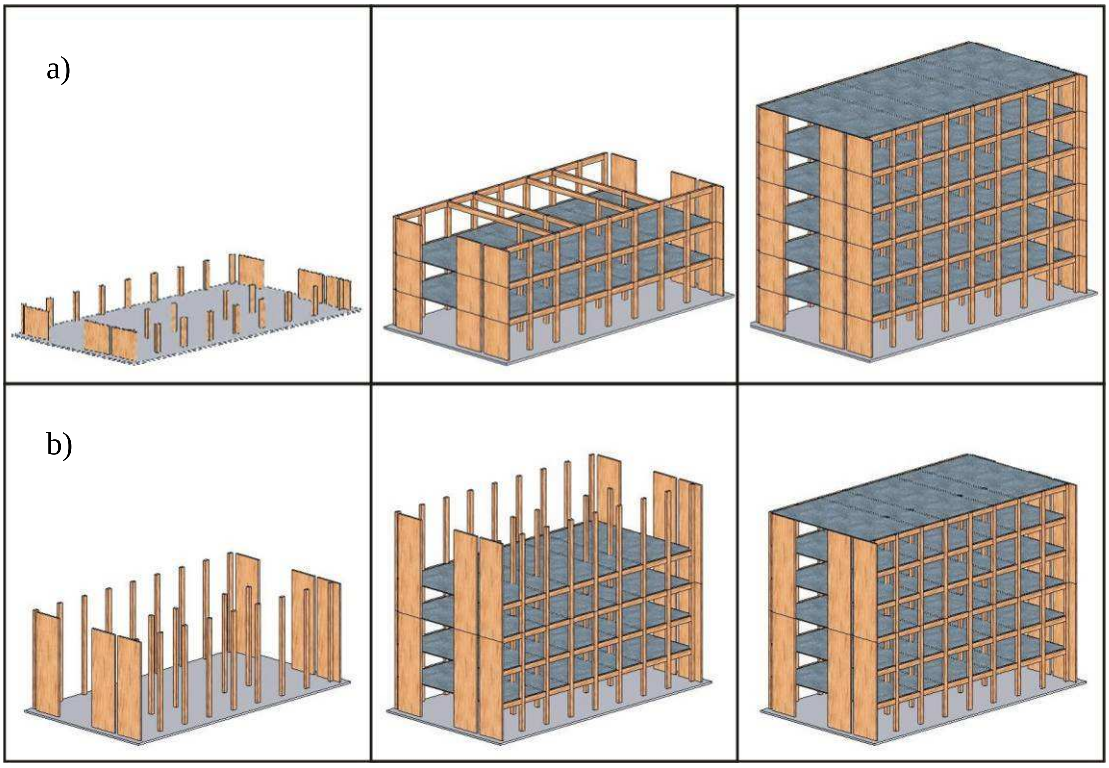


Figure 7.2: a) Platform construction method b) Balloon construction method

7.1.2 Site Clearing and Foundation Beams

Clearing of the site and its surroundings is necessary in order to start work on the building foundations. Work on the building will progress east to west in direction, working down the building. Once the site clearing of Section One is complete, the foundation trenches are dug, formwork and reinforcing is placed and the foundation is poured. Subsequently, the foundations in Section Two and Section Three are poured as work progresses down the building.

7.1.3 Level One Assembly

Once the foundation has cured sufficiently the first of the walls and columns are placed. Section One is constructed first (Figure 7.3). As the flooring is placed in this section the plywood attached too the flooring units gives the section some rigidity. By creating this stable base off which Section Two and Section Three can be supported, the amount of lateral propping required to stabilise the wall and columns members is greatly reduced. Once the flooring units for Section One are attached the Section Two and Section Three floor units are erected. Once all the sections of the first floor have been positioned the floor mesh is placed, propping is positioned and the topping is poured.

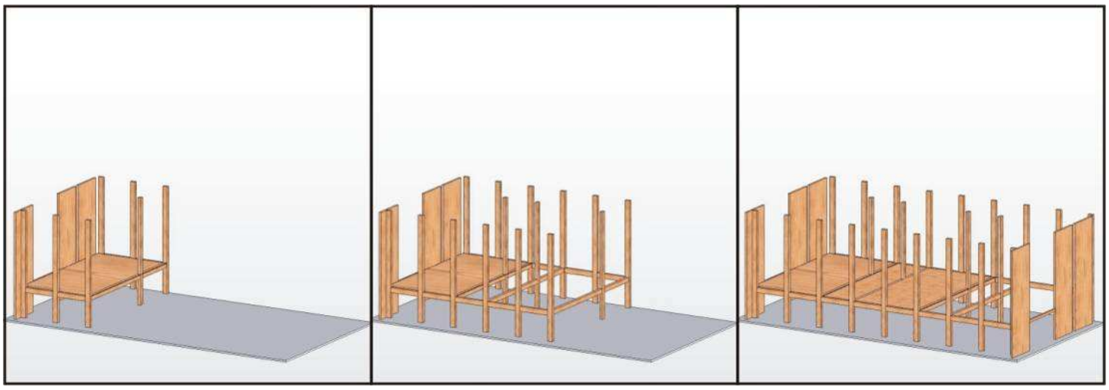


Figure 7.3: Construction of level one

7.1.4 Level Two and Three Assembly

Before completion of the level one, the second level is started (Figure 7.4), this is possible as Section Two of the first floor is placed. The assembly of level two and three proceeds in the same manner as level one. Due to the size of the floor area, two pours are necessary for the floor. It is likely that from this stage the architectural features, such as external cladding, windows and interior walls on the levels below will begin to be placed.

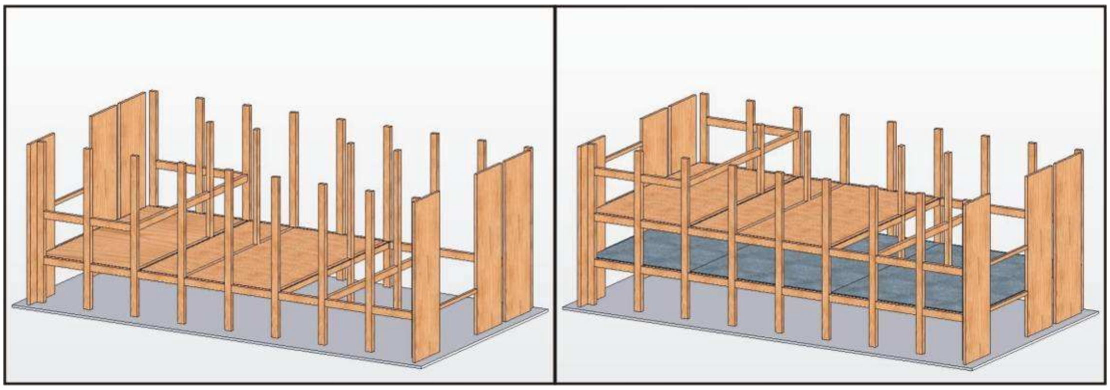


Figure 7.4: Construction of level two and three

7.1.5 Column/Wall Splicing and Level Four Assembly

On completion of Section Two of level three (Figure 7.5), work is begun on splicing the wall and column members. As with the level one assembly it is necessary to lay the flooring on Section One to create a sturdy floor section off which the remaining sections can be braced.

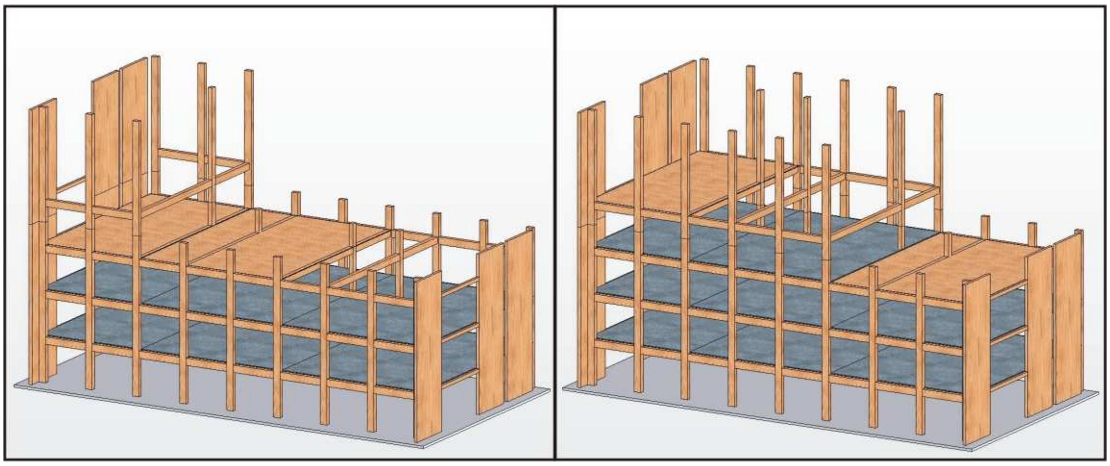


Figure 7.5: Wall and column splice and level four construction

7.1.6 Level Five and Roof Assembly

Once all of the columns and walls are spliced, construction proceeds in the same manner as the floors below (Figure 7.6). On completion of the roof level, the portal frames housing the plant are placed and final architectural fit-out is completed.

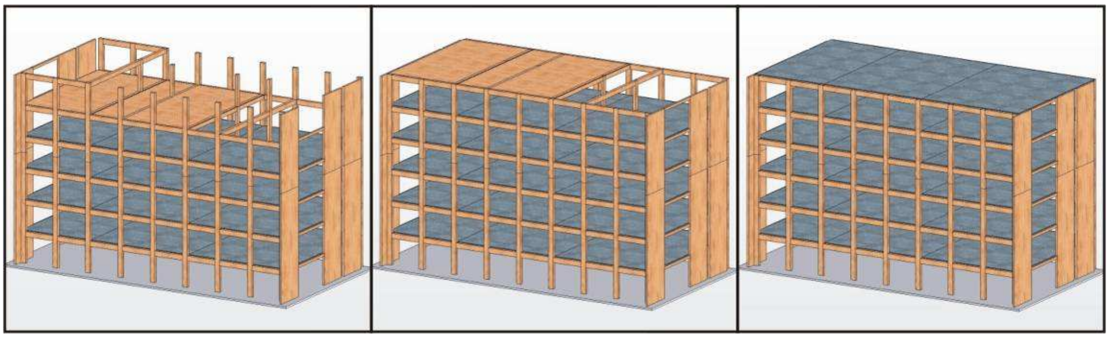


Figure 7.6: Level five and roof construction

7.2 Assembly of Key Components

7.2.1 Wall and Column to Foundation Attachment

The base of the wall member is described in Section 4.2.4. As mentioned in Section 5.5 it is undesirable to have to use epoxy on the construction site due to the nature of the adhesive. Therefore the following method is proposed and displayed in Figure 7.7:

- Walls arrive on site with bars pre-epoxied, with threaded TCM's at the end
- Deformed bars are attached into TCM
- Walls are lowered onto foundation, bars entering preformed ducts
- Bars are grouted into foundation. Required propping is added for temporary lateral loads

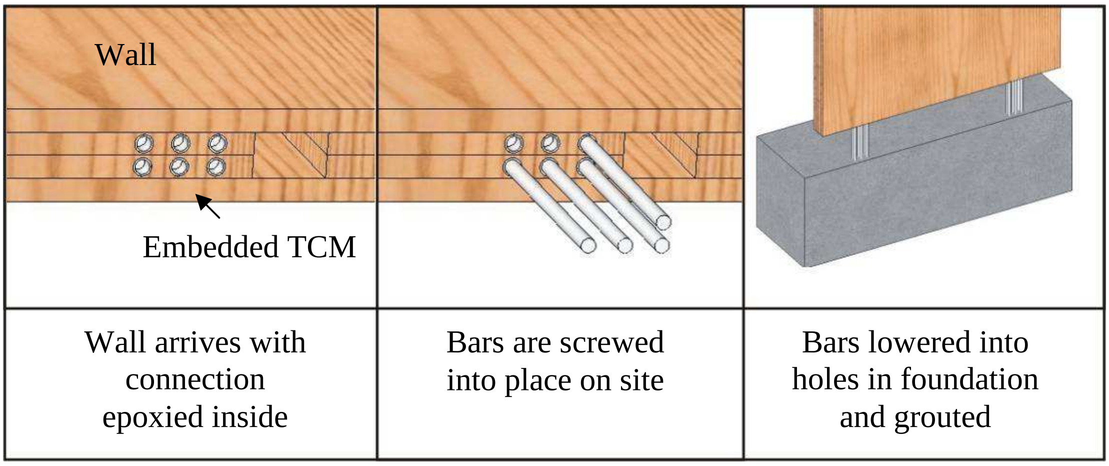
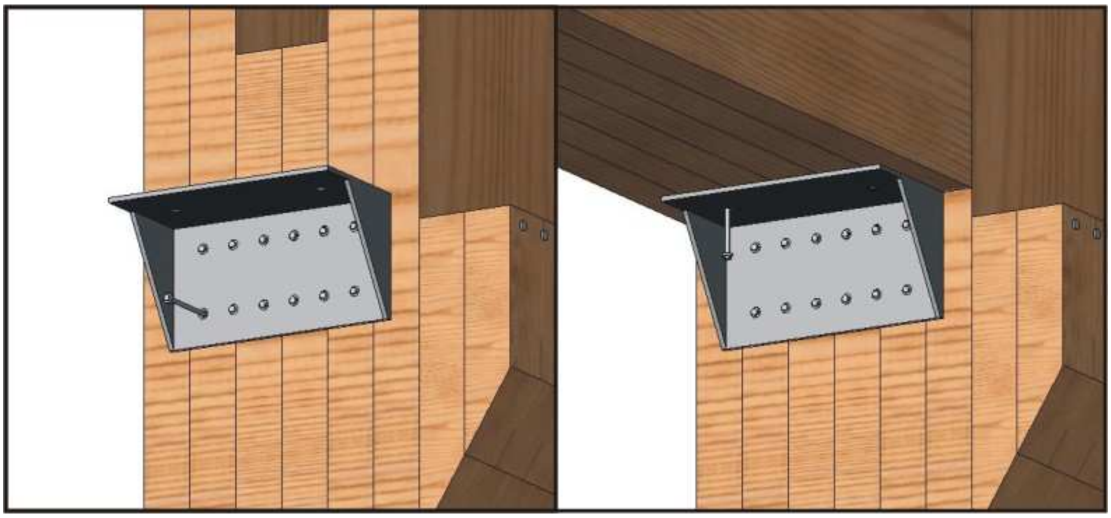


Figure 7.7: Wall to foundation assembly

Due to the nature of the column shoe foundation connection described in Section 5.5 the work required for the assembly of this component onsite is minimal. On arrival to the site the column is lifted into place and bolted to the foundation. The required propping resisting temporary lateral loads is then applied.

7.2.2 Beam Attachment

As mentioned in Section 5.3 it is required that the seismic beams be seated on corbels capable of resisting the factored dead and live gravity loading. It is logical that this corbel be utilised in the placement of the member. As displayed in Figure 7.8a the columns arrive onsite with these corbels attached. As shown in Figure 7.8b screws are placed to temporarily attach the beam. These should be removed before the post-tensioning cables are stressed to ensure the screws do not hinder the beams rocking motion.



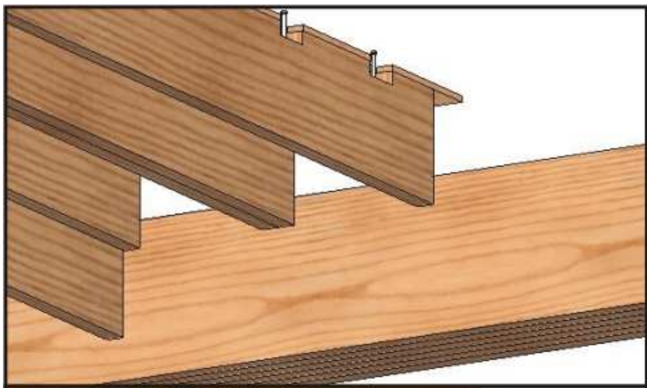
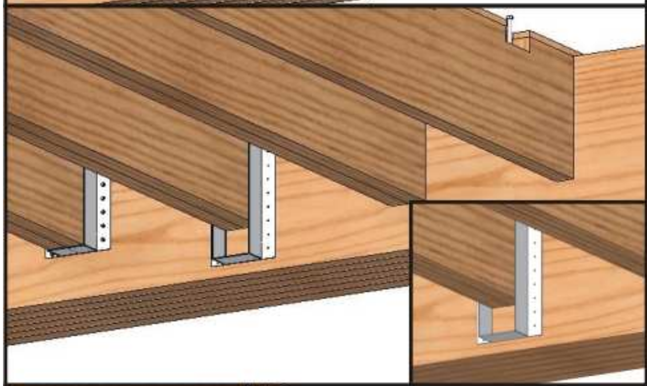
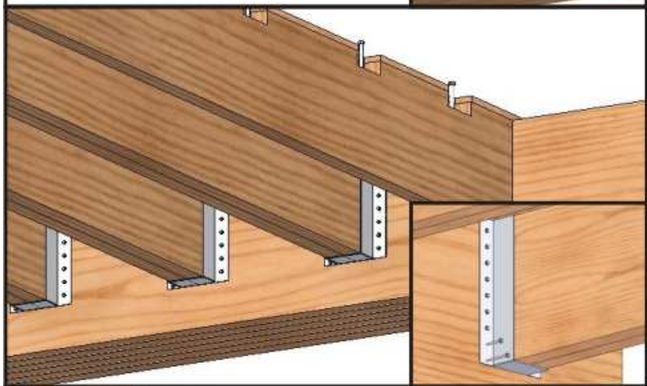
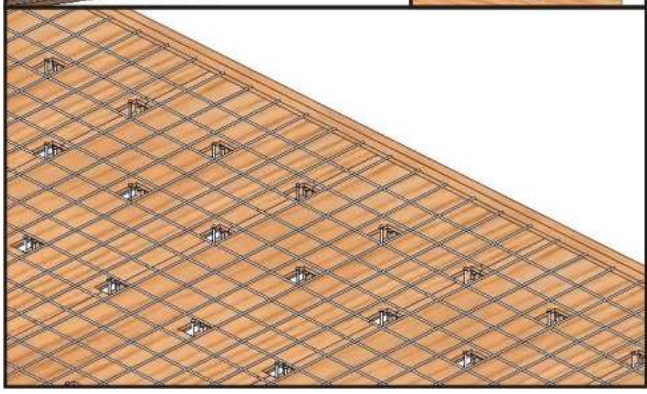
a) Corbel attached to column in factory   b) Screws placed during erection

Figure 7.8: Placement of seismic beam

7.2.3 Floor Attachment

The attachment of the flooring units described in Section 4.2.1.1 is a crucial link in the construction in of the building as it will provide rigidity to the system during construction and a platform for work in the upper storeys. The floors will arrive on site in 2.4m wide units with the plywood already attached to the joist. Table 7.1 displays the optimum form of construction for these floors.

Table 7.1: Construction of flooring units

	1. Pre-fabricated floor units are lowered into place, screws are used to attach joists to adjacent units, units may be nailed to gravity beam for stability
	2. Joist hangers are slid into place
	3. Type 17 screws are inserted into pre-drilled holes in the joist hanger
	4. Floor reinforcing is laid out, and concrete is poured

7.2.3.1 Use of Pre-topped Units

The use of pre-topped pre-cast concrete units is becoming popular in concrete construction. It is worthy to note that this may also be a good option for the timber-concrete composite flooring units. In this option each individual unit will be connected to the next with some form of discrete coupler as they are placed on-site. This will remove the need for the units to be propped during construction and may solve some of the inherent creep problems arising from the use of timber through pre-cambering as the concrete cures during casting offsite. However, there are some issue with this; a finishing topping may be required in order ensure a smooth floor surface, also cause a large increase in the cranage load of the large span floors will occur.

7.2.4 Splicing of Wall and Column Members

The six storey high wall and column members are too large to be transported to site in a single length therefore connection on site is necessary. This splicing will occur at the mid height between the 3rd and 4th levels. By splicing at mid-height the moment at the connection is kept to a minimum and the connection can be designed for shear force only. At this height the shear force in each column is 122kN with a 356kN maximum shear force occurring in each wall. The simplest method of splicing is to 'finger joint' the column/wall members together. The members are then bolted onsite as shown in Figure 7.9.

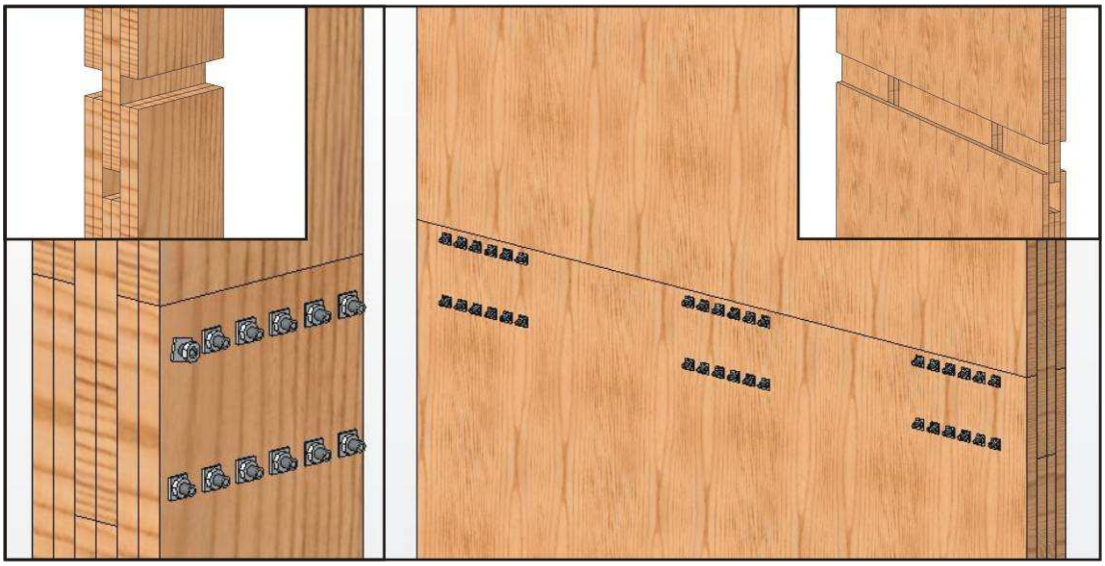


Figure 7.9: Splicing of wall and column members

This connection has been designed in accordance with NZS 3603 for a bolt loaded perpendicular to the grain.

It is noted that the closeness of fit in the joint detail may cause issues in onsite erection. With an exposed end grain the timber may swell making the connection difficult to mesh together. Therefore it is suggested that the end be tapered slightly in order to solve this problem. This can be easily done during the construction of the member and will solve the tolerance issue.

7.2.5 Post Tensioning

As shown in Figure 4.9 each beam has a cavity in which the tendons will be placed. Due to the issues relating to the corrosion of steel it is important that the tendon be placed inside ducting so that anti-corrosion protection can be applied. This also assists in guiding the tendon group through the member as it is placed. As the tendons are continuous down the length of the building the ducting will be placed as each beam is positioned, with the tendon being placed as soon as the duct is in position. Once the tendons are placed in both seismic resisting frames tensioning will occur and this specific level.

The wall unit does not use the traditional wire tendon, but instead uses threaded (MacAlloy) bars in order to achieve moment resistance (Figure 4.10). These have the advantage that cost effective couplers are available, meaning that one half of the bar can be placed after level three is completed, and second half of the bar attached as the top half of the wall is placed. The use of this bar also assists in the stressing of the walls, as the stressing force is applied through the tightening of a nut meaning a large hydraulic jack is not necessary. The use of a large hydraulic jack can cause issues in vertical members if they are to be stress from the top, which is the situation for the case study building. The tendon or bar stressing is applied at the top due to stressing from the base of the member causing a significant increase in foundation depth arising from clearance needed for the hydraulic jack.

7.2.6 Post Tensioning Anchorage

The application of the local post tensioning forces to the external timber beam to column connection poses an interesting issue in the development of the building system. Traditionally, in concrete applications, the localised anchorage forces are distributed through the use of a cone set inside the concrete member. This solution is currently not possible in a timber column and another form of attachment is necessary. These considerations lead to the development of the steel end cap shown in Figure 7.10.

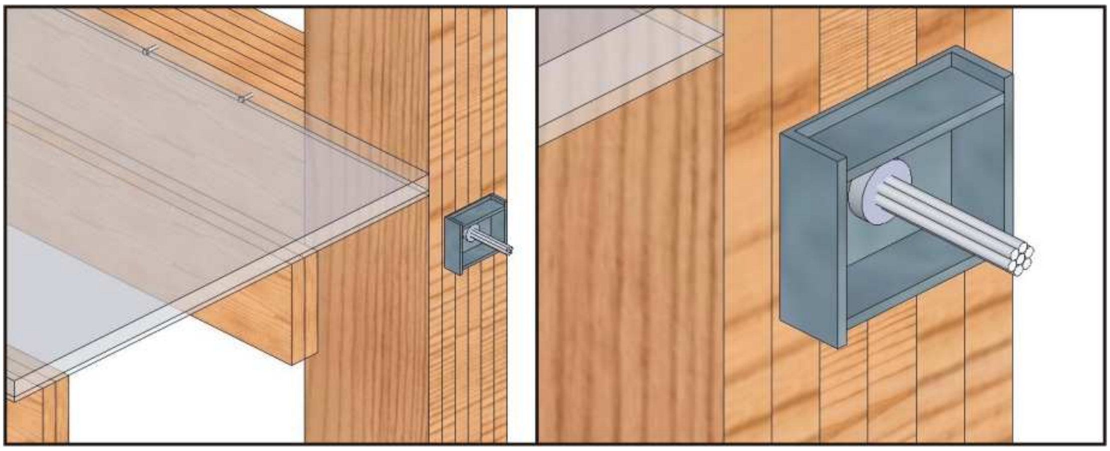


Figure 7.10: Tendon anchorage

This connection simply spreads the force over the necessary area and stiffener plates are added to increase the rigidity of the channel section. The design of this connection should take into consideration two objectives; the first is to limit the creep arising from perpendicular to grain loading, the second must consider the hierarchy of failure during a maximum credible earthquake event. The author suggests that further research must be carried out in order to fully understand the connection, and propose possible solutions.

7.3 Construction Method of Concrete Building

The construction of the alternative concrete structure would proceed in a similar manner to that of the timber structure, as both consist largely of prefabricated members. The same 'section' construction technique will be adopted. The major variation between the two buildings is that the wall and columns of the concrete structure are only of a single storey in height, meaning that platform construction rather than balloon construction will be used. Figure 7.11 shows the assembly of this structure.

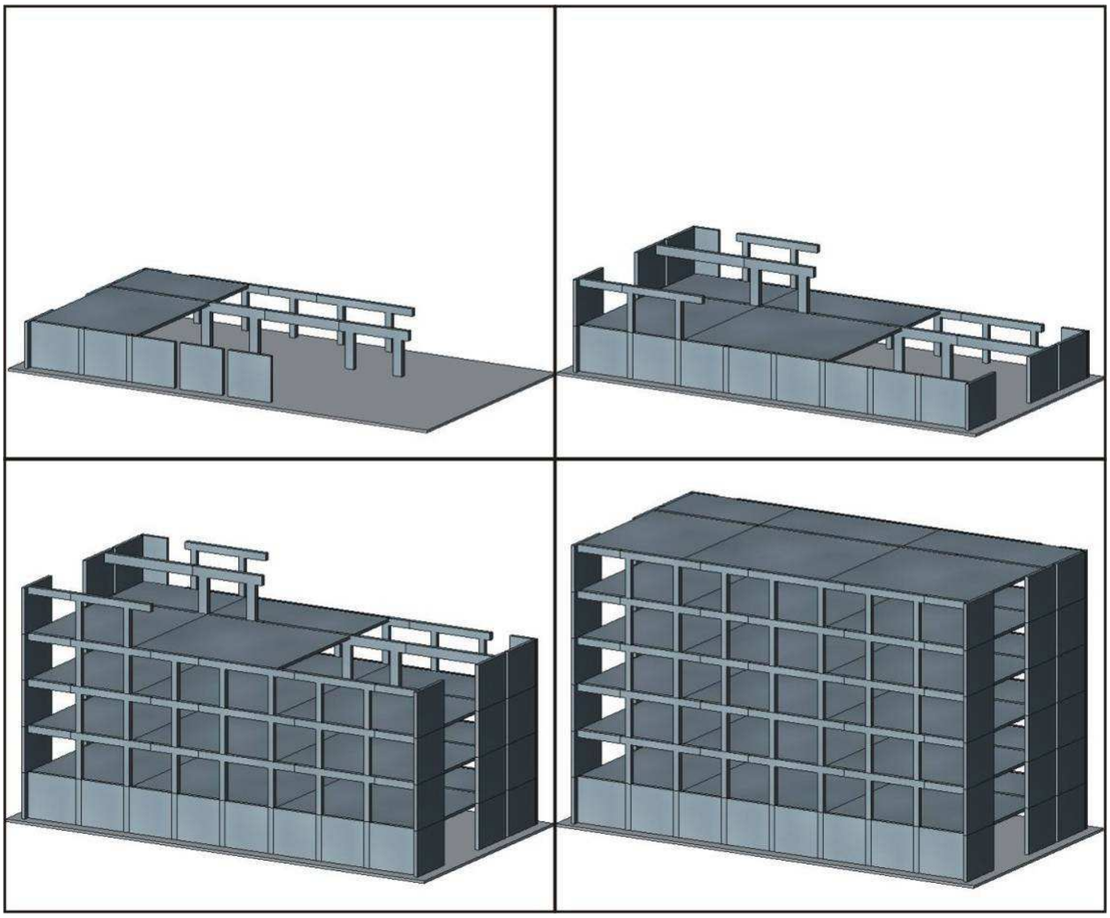


Figure 7.11: Construction of precast concrete case study building

7.4 Construction Time Analysis

The time taken on a construction project can have a considerable effect on the feasibility of a given project, therefore one of the key performance indicators of any construction system is the overall construction time. With this in mind, the time taken to assemble the case study building has been analysed and comparisons with the concrete case study building have been made. Arrow International Ltd. was consulted to ensure the construction scheduling for both case studies are estimated with reasonable accuracy.

Some assumptions had to be made in order to predict the necessary time needed, these assumptions are listed below:

- Column and wall members will take one hour to erect after arrival onsite
- Beam members will take half an hour to place after arrival onsite
- Flooring units will take twenty minutes to place after arrival onsite
- The floor topping will be undertaken in two pours, each taking one day

- Architectural fit-out will not be considered for either building
- Available personal onsite will not limit construction time

As mentioned in Section 7.1 the building is divided into sections in order to increase rapidity of construction. Using this information Gantt charts of the proposed construction sequence were developed for both the timber (Figure 7.12) and concrete (Figure 7.13). Note that S1, S2 and S3 refers to Section One, Section Two and Section Three respectively.

ID	Task Name	Duration
1	**PROJECT DURATION CONCRETE**	**67 days**
2	**Demolition**	**10 days**
3	Site clearing	10 days
4	**Substructure**	**22 days**
5	Groundwork S1	4 days
6	Groundwork S2	4 days
7	Groundwork S3	4 days
8	**Foundation Beam GL 1**	**10 days**
13	**Foundation Beam GL A**	**10 days**
18	**Foundation Beam GL C**	**10 days**
23	**Foundation Beam GL 9**	**10 days**
28	**Gravity Column Slab GL B-3,B-4**	**10 days**
31	**Gravity Column Slab GL B-6,B-7**	**10 days**
34	**Stair well Slab**	**10 days**
37	Place reinforcing for first floor slab S1	2 days
38	Poor first floor slab	7 days
39	Place reinforcing for first floor slab S2	2 days
40	Poor first floor slab	7 days
41	Place reinforcing for first floor slab S3	2 days
42	Poor first floor slab	7 days
43	Strip Formwork	4 days
44	**First floor**	**17 days**
45	**S1**	**10 days**
46	GL 1 end walls	1 day
47	S1 GL A wall panels	1 day
48	S1 GL C wall panels	1 day
49	S1 GL B Frame	1 day
50	S1 GL C Frame	1 day
51	S1 GL B beam placed	1 day
52	Insitu column reo	1 day
53	Insitu column form	1 day
54	Insitu column pour	5 days
55	**S2**	**7 days**
56	S2 GL A wall panels	1 day
57	S2 GL B Columns	1 day
58	S2 GL C Frame 1	1 day
59	S2 GL C Frame 2	1 day
60	S2 GL B beam placed	1 day
61	Insitu column reo	1 day
62	Insitu column form	1 day
63	Insitu column pour	5 days
64	**S3**	**7 days**
65	GL 9 end walls	1 day
66	S3 GL A wall panels	1 day
67	S3 GL C wall panels	1 day
68	S3 GL C Frame	1 day
69	S3 GL B beam placed	1 day
70	Insitu column reo	1 day
71	Insitu column form	1 day
72	Insitu column pour	5 days
73	Prop and cast insitu joints	2 days
74	**Flooring**	**11 days**
75	S1 Flooring	1 day
76	S2 Flooring	1 day
77	S3 Flooring	1 day
78	Prop flooring	2 days
79	Pour topping	7 days
80	**Second floor**	**15 days**
110	**Third floor**	**15 days**
140	**Forth floor**	**15 days**
170	**Fifth floor**	**15 days**
200	**Sixth Floor**	**15 days**
230	**Roof Area**	**6 days**
231	Placement of roof portal frames	2 days
232	attachement of roof joists	2 days
233	attachement of roof cladding	2 days

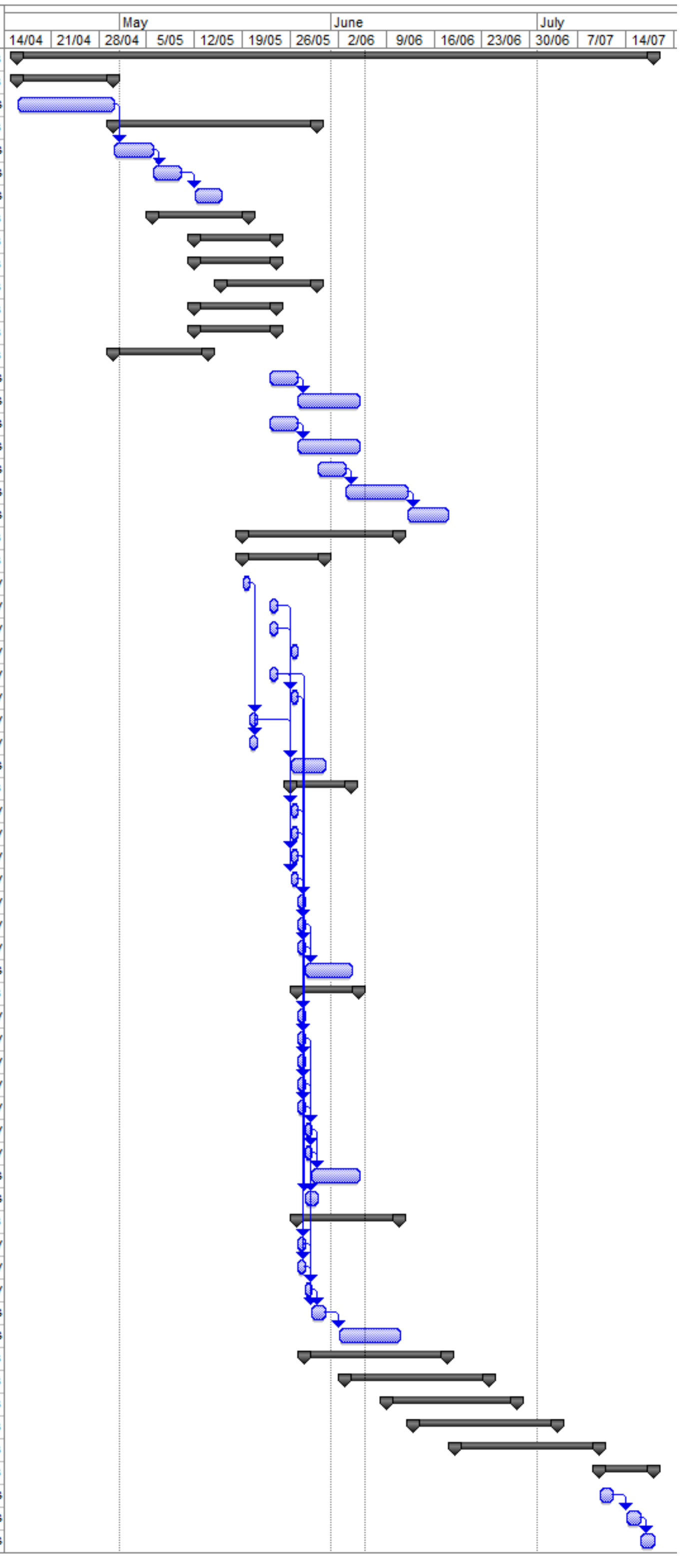


Figure 7.12: Construction schedule summary for precast concrete building

ID	Task Name	Duration
1	**PROJECT DURATION TIMBER**	**69 days**
2	**Demolition**	**10 days**
3	Site clearing	10 days
4	**Substructure**	**22 days**
5	Groundwork S1	4 days
6	Groundwork S2	4 days
7	Groundwork S3	4 days
8	**Foundation Beam GL 1**	**10 days**
13	**Foundation Beam GL A**	**10 days**
18	**Foundation Beam GL B**	**10 days**
23	**Foundation Beam GL 9**	**10 days**
28	**Gravity Column Slab GL B-3,B-4**	**10 days**
31	**Gravity Column Slab GL B-6,B-7**	**10 days**
34	**Stair well Slab**	**10 days**
37	Place reinforcing for first floor slab S1	2 days
38	Poor first floor slab	7 days
39	Place reinforcing for first floor slab S2	2 days
40	Poor first floor slab	7 days
41	Place reinforcing for first floor slab S3	2 days
42	Poor first floor slab	7 days
43	Strip Formwork	4 days
44	**First Three Storeys**	**26 days**
45	**Columns and walls**	**10 days**
46	**Siesmic Columns**	**1 day**
47	S1 Columns placed and proppec	1 day
48	S2 Columns placed and proppec	1 day
49	S3 Columns placed and proppec	1 day
50	**Siesmic walls**	**10 days**
51	GL1 Walls placed and propped	1 day
52	Pour Grout for Dissipaters GL 1	1 day
53	GL9 Walls placed and propped	1 day
54	Pour Grout for Dissipaters GL-9	1 day
55	Gravity Columns Placed	1 day
56	**First Floor**	**14 days**
57	**Siesmic Beams**	**3 days**
58	S1 Siesmic beams	1 day
59	S2 Siesmic beams	1 day
60	S3 Siesmic beams	1 day
61	Tendon Placed on GL A	1 day
62	Tendon Placed on GL C	1 day
63	Tendon Stressed GL A	1 day
64	Tendon Stressed GL C	1 day
65	**Gravity Beams**	**1 day**
66	Gravity beam 11	1 day
67	Gravity beam 12,13,14	1 day
68	**Floor Units**	**4 days**
69	Flooring for S1	1 day
70	Flooring for S2	1 day
71	Flooring for S3	1 day
72	Reo bars placed in wall	1 day
73	Floor mesh placed S1 and S2	1 day
74	Floor mesh placed S3	1 day
75	Foor propped from below S1 and S2	1 day
76	Foor propped from below S3	1 day
77	Insitu Slab Poured S1 and S2	8 days
78	Insitu Slab Poured S1 and S2	8 days
79	**Second Floor**	**16 days**
102	**Third Floor**	**13 days**
125	**Second Three Storeys**	**23 days**
126	**Columns and walls**	**6 days**
137	**Forth Floor**	**14 days**
160	**Fifth Floor**	**15 days**
183	**Sixth Floor**	**13 days**
206	**Roof Area**	**6 days**
207	Placement of roof portal frames	2 days
208	attachement of roof joists	2 days
209	attachement of roof cladding	2 days

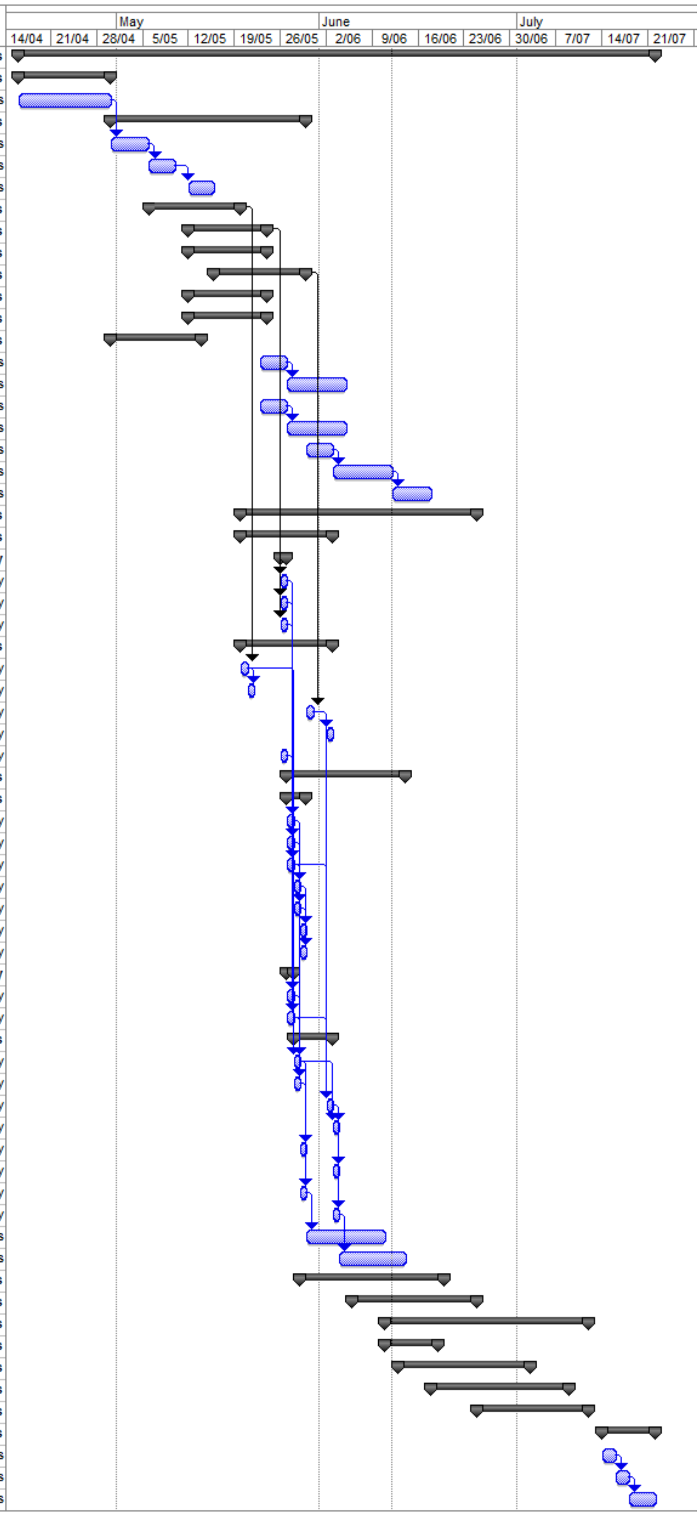


Figure 7.13: Construction schedule summary for timber building

From Figures 7.12 and 7.13 above it can be seen that the overall construction time for the Concrete building is 67 days and 69 days for the Timber building. The first floor of each structure takes the longest time as the foundations must cure adequately. It can be seen that the sub structure work takes almost one third of the time of construction in each building. On completion of the first level the rapidity of pre-fabricated construction is evident. Construction time between floors is approximately 4 days with each floor taking approximately 15 days to complete.

The major point of difference between the two buildings is the method of construction used. The use of the balloon construction method means that the Timber structure only places vertical members at two points during construction, compared to the concrete structure which must place wall and column members at each floor. The concrete assembly negates this issue by using pre-fabricated members containing both column and beam elements, and as less members are required on each floor a similar time can be achieved.

A direct comparison between the two construction times shows little difference in time meaning that comparable construction times can be achieved with the proposed post-tensioned timber construction.

8 Costs

This section will be split into two parts. The first will detail the total perceived cost of the building as calculated by Davis Langdon Shipston Davies, a quantity surveyor assisting on the project. The second part will consider the costs associated with pre-fabrication and craning and haulage of the timber system.

As describe in Chapter 3 four buildings have been designed in order to compare the characteristics of the different materials used, both structural and architectural. This chapter will compare the difference in cost between the concrete, steel, and timber structural options. The ‘Timber Plus’ structure will not be analysed in detail as the differences are predominantly architectural.

The following elements are considered in the cost analysis:

- Substructure
- Structural Frame
- Structural Walls
- Flooring (structural)
- Roof
- Exterior walls and finishes
- Windows and exterior doors
- Interior walls and doors
- Floor and ceiling finishing
- Stairs and Balustrades
- Fire protection, Electrical services and Plumbing
- Heating and Ventilation
- Vertical and Horizontal transportation
- Drainage and External works
- Sundries

A 15% margin is also applied as standard practice. Table 8.1 shows the results and comparison of the costings.

Table 8.1: Costing estimates for the concrete, steel, timber and timber plus buildings

CONCRETE		**STEEL**		**TIMBER**		**TIMBER PLUS**	
Substructure							
Frame Foundation E-W	$44,850	Frame Foundation E-W	$44,850	Frame Foundation E-W	$44,850	Frame Foundation E-W	$44,850
Wall Foundation N-S	$66,600	Frame Foundation N-S	$66,600	Wall Foundation N-S	$66,600	Wall Foundation N-S	$66,600
Gravity Raft Foundations	$9,230	Gravity Raft Foundations	$9,830	Gravity Raft Foundations	$8,080	Gravity Raft Foundations	$8,080
Lift Shaft Foundation	$8,000	Lift Shaft Foundation	$8,000	Lift Shaft Foundation	$8,000	Lift Shaft Foundation	$8,000
Total	**$128,680**		**$129,280**		**$127,530**		**$127,530**
Ground Floor Slab	$89,110	Ground Floor Slab	$89,110	Ground Floor Slab	$88,390	Ground Floor Slab	$88,390
Total	**$89,110**		**$89,110**		**$88,390**		**$88,390**
Structural Elements							
RC Columns	$133,750	Steel Columns	$348,280	LVL Columns	$407,700	LVL Columns	$407,700
RC Beams	$469,260	Steel Beams	$690,995	LVL Gravity Posts	$55,490	LVL Gravity Posts	$55,490
Lift Shaft Walls	$122,150	Steel Braces	$114,115	LVL Beams	$667,440	LVL Beams	$667,440
RC Walls	$731,500	Roof Structure	$100,448	LVL Walls	$668,150	LVL Walls	$668,150
Roof Structure	$100,448	Connections	$123,500	Roof Structure	$48,300	Roof Structure	$48,300
		Fire Protection	$126,240	Lift SHS	$13,086	Lift SHS	$13,086
Total	**$1,557,108**		**$1,503,578**		**$1,860,166**		**$1,860,166**
Dycore & Unispan Floor	$723,550	Comflor 80 Floor	$516,480	LVL Composite Floor	$688,640	LVL Composite Floor	$688,640
Total	**$723,550**		**$516,480**		**$688,640**		**$688,640**
Roofing							
Colorsteel Roofing	$73,270	Colorsteel Roofing	$73,270	Colorsteel Roofing	$73,270	Colorsteel Roofing	$73,270
Butyl Water Proofing	$69,040	Butyl Water Proofing	$69,040	Butyl Water Proofing	$69,040	Butyl Water Proofing	$69,040
Downpipes	$6,320	Downpipes	$6,320	Downpipes	$6,320	Downpipes	$6,320
Paint	$8,700	Paint	$8,700	Paint	$8,700	Paint	$8,700
Total	**$157,330**		**$157,330**		**$157,330**		**$157,330**
Exterior Walls and Finish							
Concrete Panels (Painted)	$80,160	Profiled Clad and Ext wall	$361,600	Walls and Clad (Painted)	$428,700	Walls and Clad (Painted)	$748,020
Roof Parapet	$36,000	Exterior Paint	$35,700	Plant Room Cladding	$28,920	Plant Room Cladding	$28,920

Vitro Panel (Painted)	$129,500	Plant Room Cladding	$27,715				
Plant Room Cladding	$28,920						
Total	**$274,580**		**$425,015**		**$457,620**		**$776,940**
Windows and Exterior Doors							
Glazing	$933,200	Glazing	$933,200	Glazing	$933,200	Glazing	$815,350
External Doors	$12,000	External Doors	$12,000	External Doors	$12,000	External Doors	$15,000
Total	**$945,200**		**$945,200**		**$945,200**		**$830,350**
RC Stairs & Balustrade	$72,900	RC Stairs & Balustrade	$72,900	Timber Stairs & Balustrade	$54,000	Timber Stairs & Balustrade	$54,000
Total	**$72,900**		**$72,900**		**$54,000**		**$54,000**
Interior Walls (Painted)	$447,260	Interior Walls (Painted)	$501,615	Interior Walls (Painted)	$528,160	Interior Walls (Painted)	$578,860
Total	**$447,260**		**$501,615**		**$528,160**		**$578,860**
Interior Doors	$68,200	Interior Doors	$68,200	Interior Doors	$68,200	Interior Doors	$68,200
Total	**$68,200**		**$68,200**		**$68,200**		**$68,200**
Floor and Ceiling Finishes							
Carpets and Vinyl	$361,380	Carpets and Vinyl	$361,380	Carpets and Vinyl	$361,380	Carpets, Vinyl, Timber floor	$367,410
Roof Tiles & Paint	$230,950	Roof Tiles & Paint	$230,950	Roof Tiles & Paint	$230,950	MDF roof Tiles & Paint	$346,780
Total	**$592,330**		**$592,330**		**$592,330**		**$714,190**
Plumbing	$67,400	Plumbing	$67,400	Plumbing	$67,400	Plumbing	$67,400
HVAC	$1,382,700	HVAC	$1,382,700	HVAC	$1,382,700	HVAC	$1,382,700
Fire Protection	$345,675	Fire Protection	$345,675	Fire Protection	$345,675	Fire Protection	$345,675
Power and Lighting	$599,170	Power and Lighting	$599,170	Power and Lighting	$599,170	Power and Lighting	$599,170
Lift	$200,000	Lift	$200,000	Lift	$200,000	Lift	$200,000
Data and Comms System	$50,000	Data and Comms System	$50,000	Data and Comms System	$50,000	Data and Comms System	$50,000
Paving and Drainage	$54,840	Paving and Drainage	$54,840	Paving and Drainage	$54,840	Ceder paving and Drainage	$58,800
Aluminium Louvre	$391,600	Aluminium Louvre	$391,600	Aluminium Louvre	$391,600	Timber Louvre	$427,200
Maintenance	$55,200	Maintenance	$55,200	Maintenance	$55,200	Maintenance	$55,200
Total	**$3,146,585**		**$3,146,585**		**$3,146,585**		**$3,186,145**
GRAND TOTAL	$8,202,833		$8,147,623		$8,714,151		$9,130,741
GRAND TOTAL + MARGIN	$9,433,258		$9,369,766		$10,021,274		$10,500,352

As shown in the Table 8.1 the timber building would cost approximately $600,000 (6% of the total cost) more than the concrete and steel structures. It is also evident that a lot of the architectural features of the structure were kept the same (with some changes required for durability).

The first item of cost difference shown is that of the difference in the substructure. It can be seen that this difference is negligible. As described in section 4.3 the foundations are similar due to the nature of the structure, with a significant amount of the foundations requiring overturning resistance due to seismic loading. The small saving is due to the lighter nature of the timber structure.

It is clear that the major cost difference between the structures is that of the structural elements and flooring system. The Timber structural system (frame, wall, and gravity) is calculated to cost $360,000 (24%) more than that of the steel option and $300,000 (19%) more than the concrete systems. These are significant differences and represent the major portion of the buildings overall cost differences. The composite flooring system is calculated to be $689,000 which is less than the concrete flooring ($724,000) but greater than the steel ($516,000).

Significant cost savings occur in comparing the wall cladding necessary in the timber, concrete and steel structures. The use of the 'thermomass' panels for the concrete building means that cladding is not needed. In comparison the steel and timber structural walls must be protected, thus cladding is paramount. This adds significant cost to the structure with extra costs of $360,000 and $430,000 to the steel and timber buildings respectively.

The additional timber architecture of the Timber Plus building adds $480,000 to the cost of the Timber Building. Showing a 11% increase in total cost when compared to the concrete and steel structures.

Costing of these elements has been performed using estimates and calculations from previous jobs. Factors are applied to a beam if it is to be post-tensioned. To allow for the fabrication of the member an additional cost per m^3 is added.

8.1 Cost of Prefabrication

It is noted that two major element costs differentiate the price of the timber building from that of both the concrete and steel buildings. These costs are represented by the timber-concrete composite flooring and the large structural timber elements. These elements also represent the greatest uncertainty in the costing as it is difficult to conclude an “in place” cost of any new system.

Although it is clear that the largest cost in these elements is the timber itself, with both LVL and Plywood being relatively expensive per m^3 when compared to concrete, the cost of member prefabrication also represents a large proportion of the total cost. Initially a cost of \$200 per m^3 was used for the fabrication of the timber members. This value is based of the fabrication cost of previous large scale Glue Laminated beams. On personal communication with Carter Holt Harvey Limited, a value of \$500 per m^3 was quoted (Banks 2008) further increasing the overall cost of the building by \$200,000. This highlights a major gap in the production of this type of building as although this price was quoted, it was also stated that Carter Holt Harvey did not necessarily posses the necessary equipment needed to manufacture these members efficiently (in both time and cost). Considering this it is suggested that this gap in the supply of the system must be addressed in order for the method of construction to become truly viable.

8.2 Craning and Haulage Cost

One of the major advantages in the usage of timber is the lower density of the product when compared to concrete. It is shown in Table 8.2 that the sizes of the members remain similar for the timber and concrete case study buildings. Table 8.2 shows the weight of these members.

Table 8.2: Member Weight comparison between timber and concrete buildings

	Timber		**Concrete**	
	Dimension (mm)	Weight (T/m)	Dimension (mm)	Weight (T/m)
Beam	600 x 378	0.14	800 x 400	0.78
Column	600 x 378	0.14	800 x 400	0.78
Wall	4000 x 252	0.63	4300 x 200	2.10

It can be seen in Table 8.2 that considerable weight savings occur in the timber members which will have a significant effect on the transportation costs. One of these savings is the cost of placement of the members on site. Titan Cranes Limited (Titan Cranes Limited 2008) charge out cranes on a per hour bases and these values are used to indicate the possible savings arising from the lighter members. The largest member placed onsite in the timber building is that of the wall unit with a total weight of 8.3 tonnes this will require the use of a crane with a 10 tonne capacity at an average cost of $140/h. The largest concrete member to be placed is also the wall unit at a weight of 8 tonnes; therefore on site the same crane can be used. Although this is the case for this specific building the walls in the concrete building are only of one storey in height. If the concrete walls were of three storeys (as in the timber building) the weight would increase substantially to 28 tonnes requiring a significantly larger 55 tonne crane at a cost of $300 per hour. Further to this saving haulage costs are often based on weight, therefore large cost savings can be realised.

This study has not involved detailed consideration the ‘onsite’ costs involved in construction (e.g. numbers and required skill level of personal), instead general ‘as built’ values were used. It is important that further investigation be performed in order to understand additional cost/possible savings involved in the onsite operations involved in the construction of a post-tensioned timber building.

9 Business Concept

Often when the future of a research project is discussed the main focus is placed on purely research based objectives. Although these considerations are important it is also crucial that the commercial future of the system is also considered. The following section aims to discuss the way forward for the post-tensioned hybrid timber system in the construction market, a market which is exceedingly hard to break into due to competitive conditions, slow adaptation of technology, product saturation and low profit margins.

For any new product to enter an already existing market it must have competitive advantage over the existing products in that same market. The key advantages of this system are listed below:

- A simple moment resisting connection has been developed for both wall/column to foundation and beam to column joints: this means lateral force resistance is possible, previously large timber moment connections have proved to be complicated and costly.
- Open floor plans can be achieved: with the use of perimeter lateral resisting elements and long span flooring this method of construction is useable not only in residential and hotel type structures but also for large office structures.
- Low damage during seismic events: due to the nature of the hybrid connection little or no damage will occur in the building after a major seismic or wind event. This means that the building not only eliminate loss of life but also be available for immediate occupancy eliminating the downtime of the structure.
- Increase fire resistance: the fire resistance of large timber members is well proven (Lane 2004). The integrity of large members will remain during charring and the strength of the residual section can be checked.

Although these advantages are considered to be very important a product often needs a single strong selling point that can be used to get that product into the market. In the case of the hybrid timber connection, the fact that it is timber will give this product its key competitive advantage. Sustainability is becoming increasingly important in modern society with emphasis being placed not only bottom line costs but also on environmental costs. Carbon has become a form of measurement of environmental impact and timber is the only material in the world with a negative carbon output. Further to this as fuel costs rise the fact that LVL is grown and manufactured in New Zealand provides a key advantage to the use of timber in construction. Therefore, the clean, home-grown image of timber is paramount to the systems success.

It is recognised that the construction market worldwide is already well established and highly competitive. Therefore it is tough for any new innovation to establish itself. Porter's five forces framework (Porter 1979) can been used when making a qualitative assessment of a systems position in a given market (Figure 9.1). In this model 5 forces are defined, three from horizontal competition and two from vertical competition. The first of these to be discussed is that of the threat of new entrants, as this product is a new entrant into the market it is considered that this is not really applicable in this case. The second horizontal force is the threat from substitute products.

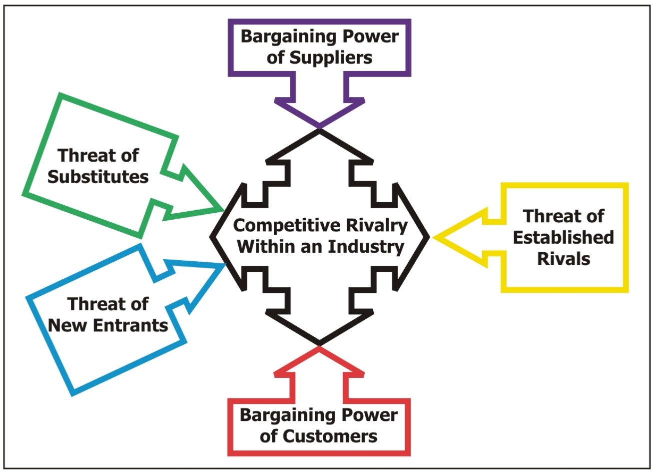


Figure 9.1: Porter's 5 forces model (Porter 1979)

Other multi storey timber solutions must be considered in this (i.e. the threat of cross laminated construction currently emerging in Europe) it is therefore important the advantages over other timber construction methods be recognised as highlighted above. The last horizontal force (threat from established forces) is undoubtedly the largest obstacle that must be overcome if the system is to gain a market share. As mentioned the construction industry is already well established and competing material industries such as concrete and steel are likely to actively resist any change that will lead to a decrease in these products market share. This highlights the need for timbers "clean, green, and home-grown" image to be utilised in comparison to other materials. Studying the vertical forces, the bargaining power of the customer is hard to judge however in the authors experience there is a high level of willingness to adopt a multi storey timber system although a lack of knowledge causes reluctance.

This leads directly to the second vertical force; the bargaining power of the supplier. Already Section 8.1 indicates a considerable gap in the supply chain of the system when considering the prefabrication of members, however, further gaps can be recognized if the total chain of construction is considered. Firstly it can be seen that there is a lack of knowledge in the industry about the system in general, with architects unaware that a feasible option exists for the construction of multi-storey open plan timber structures. Further to this, a lack of knowledge from design engineers leads to engineering practitioners being reluctant to implement such a system, perhaps even attempting to convince a client or architect to adopt a more common solution that they feel more comfortable in designing. For this reason the importance of improvement in the communication of design method is highlighted. Lastly the application of the post tensioning can be considered a weakness in the development of the connection due to monopoly that a few companies hold over the application of this technology leading to high costs. As mentioned above, the construction time of the system is comparable to that of pre-cast concrete and the costs of the case study (steel and concrete) buildings are also similar. Although these two conclusions have been drawn the distribution of this information is vital.

As the timber post-tensioned system is in conflict with existing and dominant technologies it can be described as a disruptive force (Bower 1995) in the market, meaning that it will aim to displace the current dominating products. Although this description is accurate the term Innovative Technology is perhaps more appropriate as the product being introduced is of a higher performance compared to existing products. Innovative (or disruptive) technologies are not necessarily disruptive to the customer and often take a significant amount of time until the established products are displaced. Christensen (1997) states that often even if an innovative technology is recognised, existing businesses are often reluctant to take advantage of it, as it would involve competing with their existing (often more profitable) technological approach. It is therefore important for the innovator to recognise a small niche market in which the product can be nurtured.

In the case of the post-tensioned timber concept this niche market has been recognised as being that of 3 and 4 storey government owned structures. Assisted by legislation stating that for all government buildings of less than four storeys must consider a timber construction option (Anderton 2007) this market is considered to be the ideal place in which the system can grow. Although it is likely that these buildings will not be of a prestigious nature, it is however likely that Government will want to advertise the use of timber further increasing public knowledge of the possibilities using post-tensioned timber. This will also provide the opportunity for the design knowledge base to be grown ensuring that the design engineers feel comfortable designing using this innovative new system. The increased cost of the structure is also of less importance to a government client placing a higher emphasis on the "clean, green and home grown' nature of the system.

Another sector that will possibly over look the slightly higher cost of the system is that of a 'prestige' client. This client will aim to use the building as a marketing tool, heavily promoting the nature of timber as a sustainable material. This high publicity will further increase knowledge and make the system more desirable to certain clients aiming to project the green image that the use of timber brings.

10 Conclusions

This thesis aimed to assess the feasibility of post-tensioned timber building. This assessment was performed with the use of case study buildings designed in timber, concrete and steel. The structural design of the timber building was presented with emphasis placed of the connection design. Subassembly testing was performed to investigate the performance of a selection of critical connections. Finally, the costs, construction technique and construction time was compared between the case study buildings and a business case was presented suggesting the way forward to ensure adoption of post-tensioned timber buildings in the construction marketplace.

Section 1.1 proposed a series of questions that the research was aiming to answer. These questions can be divided into two major categories: questions about the design of post-tensioned timber structures, and questions about comparative performance between the timber system and other common systems in steel and concrete. The answers to these questions are detailed below:

10.1 Member and Connection Design of Timber Building

How will a timber post-tensioned building be designed?

- **How will lateral seismic loading be calculated?**
  The lateral seismic loading of the building is assessed using a modified version of the Direct Displacement Based Design Procedure. Simple modifications can be made to allow for both the anisotropic and flexible nature of timber.
- **What type of flooring will be used?**
  Timber-concrete composite flooring is used. These units consist of concrete topping poured onto plywood sheets which sit on plywood joists. Notches are cut into the joist units providing composite behaviour. Diaphragm action is achieved through the concrete topping. The use of notched connections can also be used to control the deflections of gravity dominated beams. The

prefabricated nature of the flooring units means that they do not hinder construction time.

- **How will lateral forces be resisted?**

  Lateral resistance is provided using the ductile post-tensioned timber connection. This system combines the use of post-tensioned steel elements with the use of sacrificial yielding elements. Design of these connections follows the procedure for the design of a concrete ductile post-tensioned connection. A few modifications are necessary to allow for the differing stiffness values in the LVL perpendicular and parallel to the grain. It is also necessary to account for a reduced connection modulus if the perpendicular to grain stiffness has a significant effect (i.e. in a non-armoured beam to column connection)

- **What type of connections will be used and how will these connections be designed?**

  The principle aim of the connection design was simplicity. Joist hangers are currently common in practice and can also be used for the composite flooring. Bearing is used for gravity load transfer due to its simplicity and ease of design. The floor diaphragm is connected to the seismic elements through the use of discrete connectors cast into the topping. The design of these connections largely follows current code previsions.

How will these connections perform under lateral loading?

A series of testing was performed to assess the performance of key connections. pushout and subassembly tests were devised to find the performance of the connections under lateral loading. The conclusions from these tests are listed below:

- **How will shear at the base of a beam to column or wall to foundation connection be resisted without effecting the rocking motion of the member?**

  The shear sliding at the base of a wall or column is resisted using shear keys. Angled shear keys at the base of a wall or column are preferable to half circular shear keys as they reduce stress concentrations and damage. These do not affect the moment capacity of the wall or column.

- **How will floor shear be transfer to the seismic frame?**

The floor diaphragm is connected to the frames and wall using discrete connectors. The coach screw connection used in the frame direction was tested. A minimum characteristic strength is suggested for the connection, however, larger values may occur if a sudden slip failure occurs.

- **How will the placement of armour at the beam to column interface influence the moment response of the section?**

  The placement of steel armour at the beam to column interface causes a significant increase in the moment capacity of a beam to column connection by reducing the effect of the perpendicular to grain stiffness.

- **Is the predicted performance of a beam to column connection using current design procedures accurate?**

  The design procedure suggested in Newcombe et al. (2008) describes the method used to calculate the moment capacity of a post-tensioned connection subjected to a given drift. Testing has shown that this procedure adequately predicts the moment capacity of a beam to column connection.

- **Is it necessary to place corbels under the seismic beams?**

  It is currently suggested that corbels be placed under the seismic beams of a precast concrete post-tensioned frame (NSZ3101: Appendix B), neglecting the shear capacity provided by friction at the interface. Therefore it is also suggested that these corbels be placed under the LVL beams. The placement of corbels on the underside of the beam does not affect the moment response of the beam to column connection

- **How will the beam to column connection perform with the addition of a floor unit?**

  The placement of a floor unit on the beam to column subassembly caused tolerable unsymmetrical hysteretic behaviour to occur (from damage to the floor) due to the unsymmetrical nature of the specimen.

10.2 Comparisons between Timber and Other Construction Materials

How will a timber post-tensioned structure compare to the current steel and concrete structural design practice?

- **How will member size compare between the timber and concrete structures?**

  Timber member sizes are comparable to that of the concrete case study building. As timber is less dense than concrete, the timber members are significantly lighter.

- **How will construction method and construction time compare between the timber and concrete structures?**

  The construction method is performed with an aim to maximise the advantages gained from the use of prefabricated members. The construction of key components is performed aiming to maximise the amount of work performed off-site. A construction time comparison was performed between the prefabricated timber building and the precast concrete structure. This comparison showed the time needed to assemble both buildings is the same as both systems aim to maximise off-site fabrication.

- **How will the costs of the timber, concrete and steel buildings compare?**

  The total as built cost of the timber, steel, concrete and timber plus buildings was assessed. This showed the Timber building to be $600,000 (6%) more expensive than the concrete and steel options. The timber plus building (with the addition of predominantly timber architectural features) is 1,080,000 (11%) more than concrete and steel.

How will the development of the post-tensioned timber system progress in the construction market?

The overall business case for the post-tensioned timber construction system was also assessed. It is suggested that for the system to succeed, a key point of difference must be promoted. For this system that point of difference is the clean, green nature of timber. It is also important to ensure that the home grown origin of the materials used is recognised. The use of government structures is an important first step in the systems development helping architects, design engineers, and the construction industry become comfortable with the design and construction aspects of this type of building. In will then be important to establish a ‘prestige’ client, one who is willing to accept possible risks involved in the use of a new system. This building can then be used as a ‘flag ship’ structure further promoting the development of the use of post-tensioned timber moment connections both in New Zealand and around the world.

In conclusion, the use of the ductile post-tension timber connection is a feasible way to achieve long span open plan timber structures.

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Appendix A: Case Study Steel Design

STEEL CONSTRUCTION
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SCNZ ref.: s260507

Mr Nicolas Perez Fernandez
Victoria University of Wellington
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Wellington

29th May, 2007

Biological Science Building in University of Canterbury

Dear Nicolas,

As requested we have prepared alternative steel solutions for the University of Canterbury Biological Science Building. This report includes a discussion of preliminary design options developed along with sketches for each option.

Assumptions

- Soil type D in accordance with NZS 1170.5
- Superimposed Live Load 3.0 kPa
- Superimposed Dead Load 0.5 kPa

Preliminary Design Options

Two alternative floor systems are presented.

Option 1: 0.9 Comflor 80
- ~~150mm thick~~
- ~~Total floor depth, floor plus beam, is typically 610mm.~~
- ~~Spans generally 4.922m and 4.362m~~
- ~~One line of props required for 4.922m span~~
- ~~310mm thick "Thermomass" precast wall panels~~

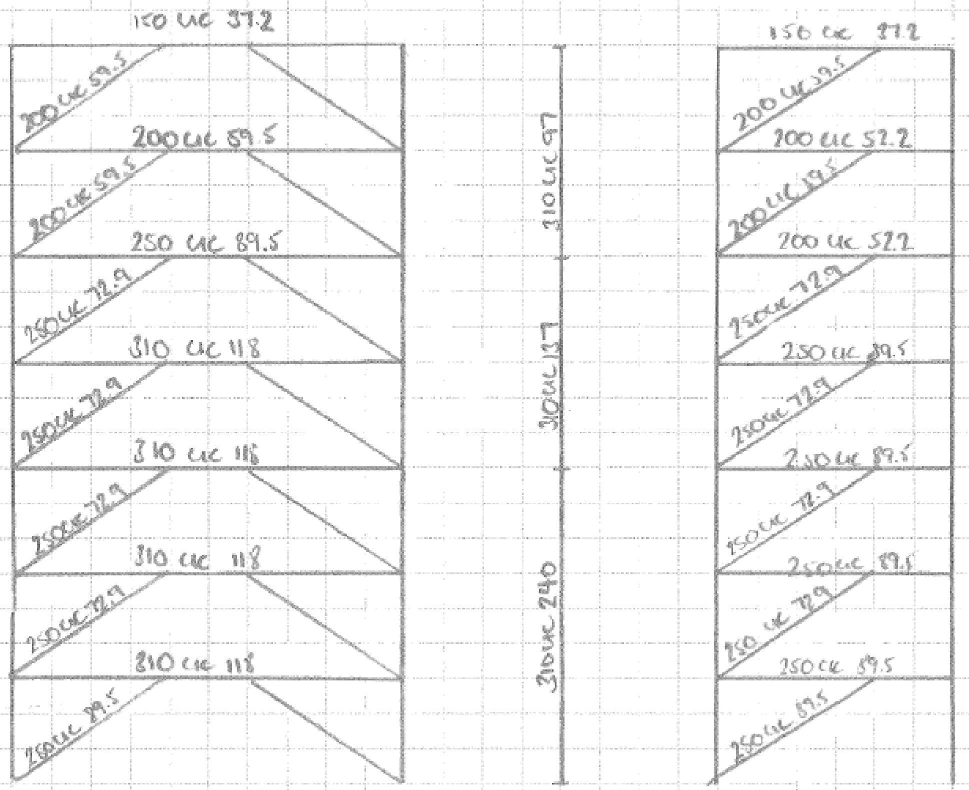
Option 2: 0.9 Comflor 80 USE OPTION 2.
- 150mm thick
- Total floor depth, floor plus beam, is typically 610mm
- Spans generally 4.922m and 4.362m
- One line of props required for 4.922m span
- 150mm thick light timber thermal isolation wall

General Benefits

UNIVERSITY OF CANTERBURY
Te Whare Wānanga o Waitaha
CHRISTCHURCH NEW ZEALAND

Project: Steel BS building.
Subject: –
By: Xian, Rex T Smith
Date: 19/11/07. Page___ of ____

Civil Engineering Department

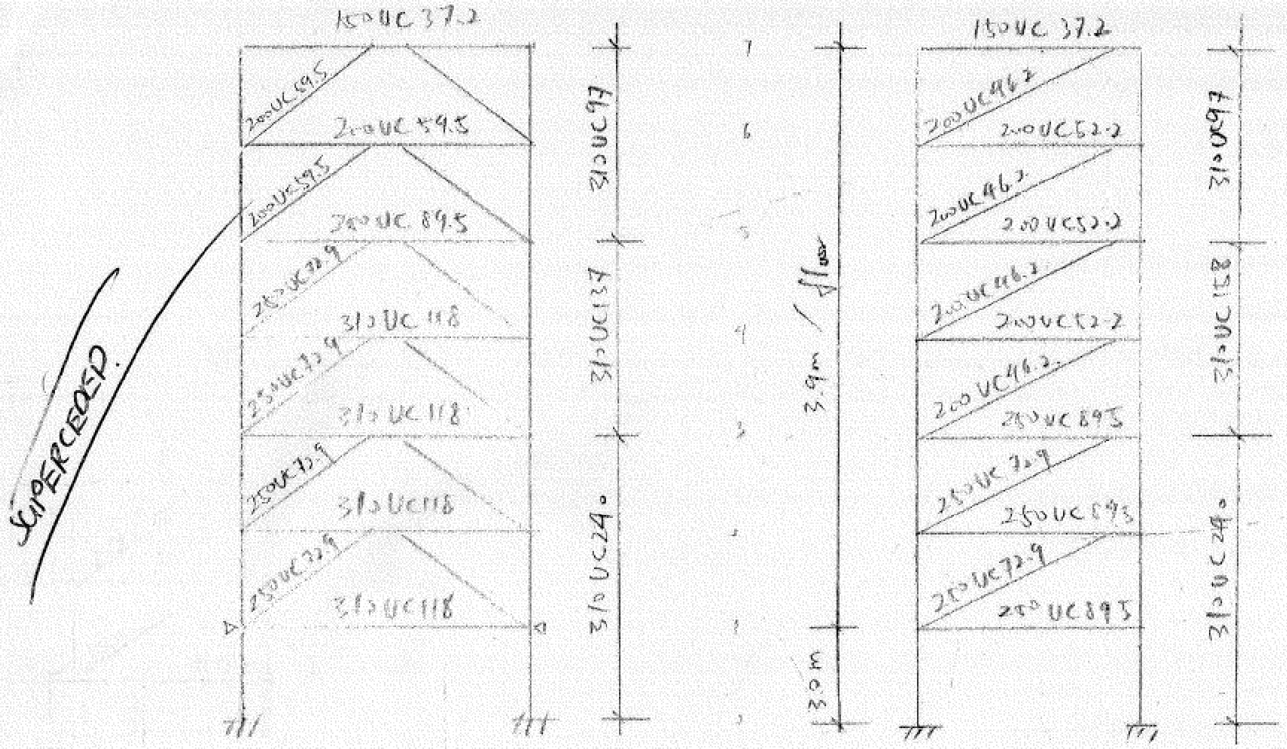


EBF1 EBF2.

Steel Structures Analysis Service

Project name Canterbury BS Building | Project no. 73 | Page 14

Subject EBF / D Braces | Date 24/05/2007 | By Xiao



Lateral System

The first floor has been assumed to be braced by the partially buried perimeter concrete shear walls and the concrete slabs on grade between Grid C and E. The floors above level 1 are braced by Eccentrically Braced Frames (EBF). There are two frames in the along direction located at the perimeter of the building. There are four frames in the transverse direction inside the building. These have been designed for a ductility $\mu = 3.0$.

The precast Thermomass panels have been assumed to be non-structural elements. These will need to be suitably detailed to accommodate interstorey deflection.

Fire Design

Steel columns will typically need 60 minutes FRR which can be provided by linings. The perimeter beams and primary beams are required to be fire rated. These could be provided by sprayed on cementatious coatings. Only a limited number secondary beams would require passive fire protection if the slab panel method for fire design is used. See attached plan for detailing of the additional slab reinforcing. A fire engineering report would be needed to confirm specific requirements.

Conclusion

Steel options have been developed for the project which have some benefits such as quick erection, reduced column loads and reduced craneage.

The Option 1 uses larger steel beams under the thermal isolation walls than the Option 2 due to the higher density of 310mm thick "Thermomass" precast wall panel than 150mm thick light weight timber thermal isolation walls.

If you have any questions please do not hesitate to contact myself on (09) 262 6685 or Alistair on (09) 262 6683.

Kind Regards,

Xiao Huantian

Huantian Xiao
BE(Civil) MEngSt ME GradIPENZ
Structural Design Engineer

Steel Construction New Zealand

A Fussell

Alistair Fussell
BE(Hon), ME(Civil), MIPENZ,
CPEng
Senior Structural Design Engineer
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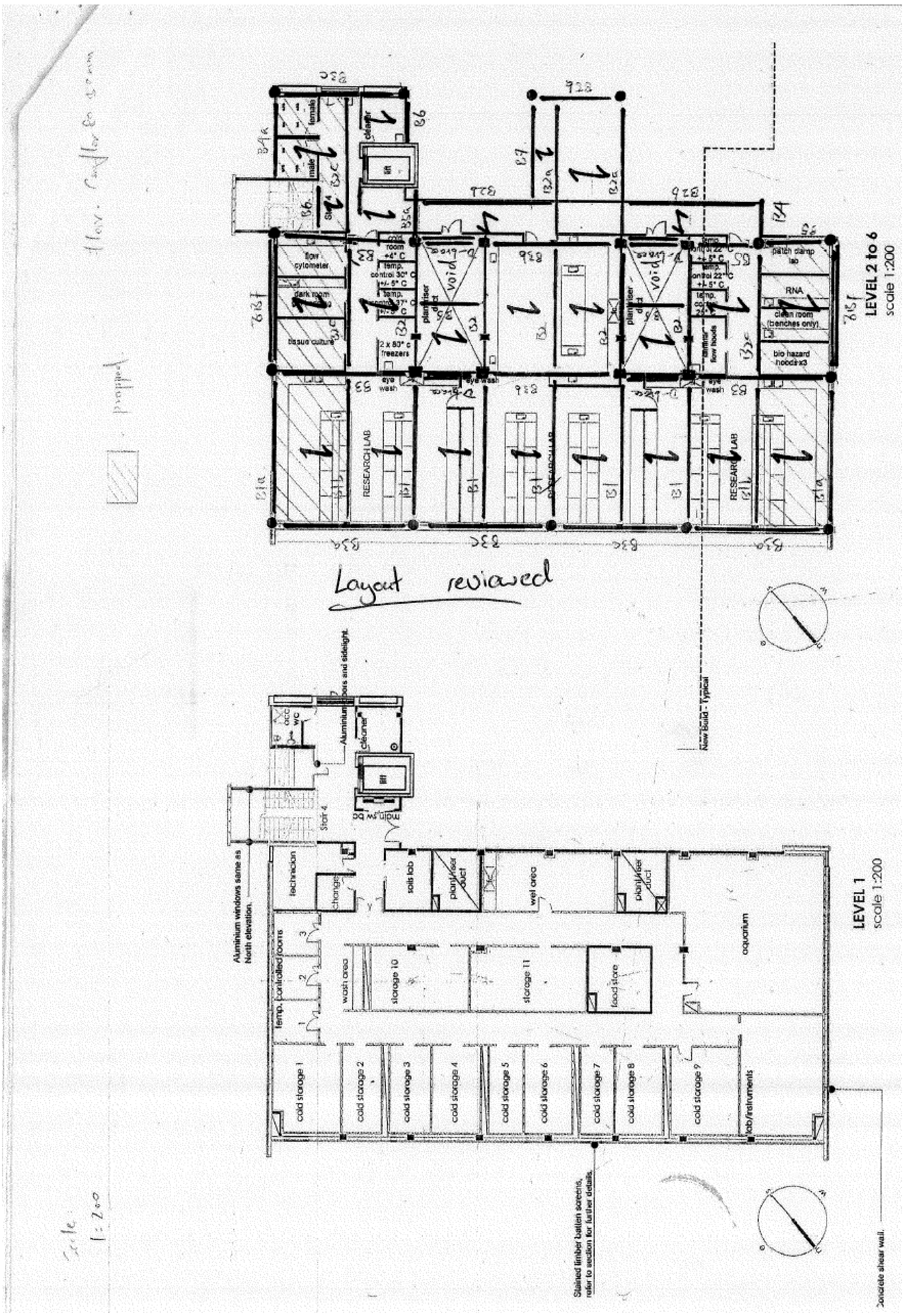
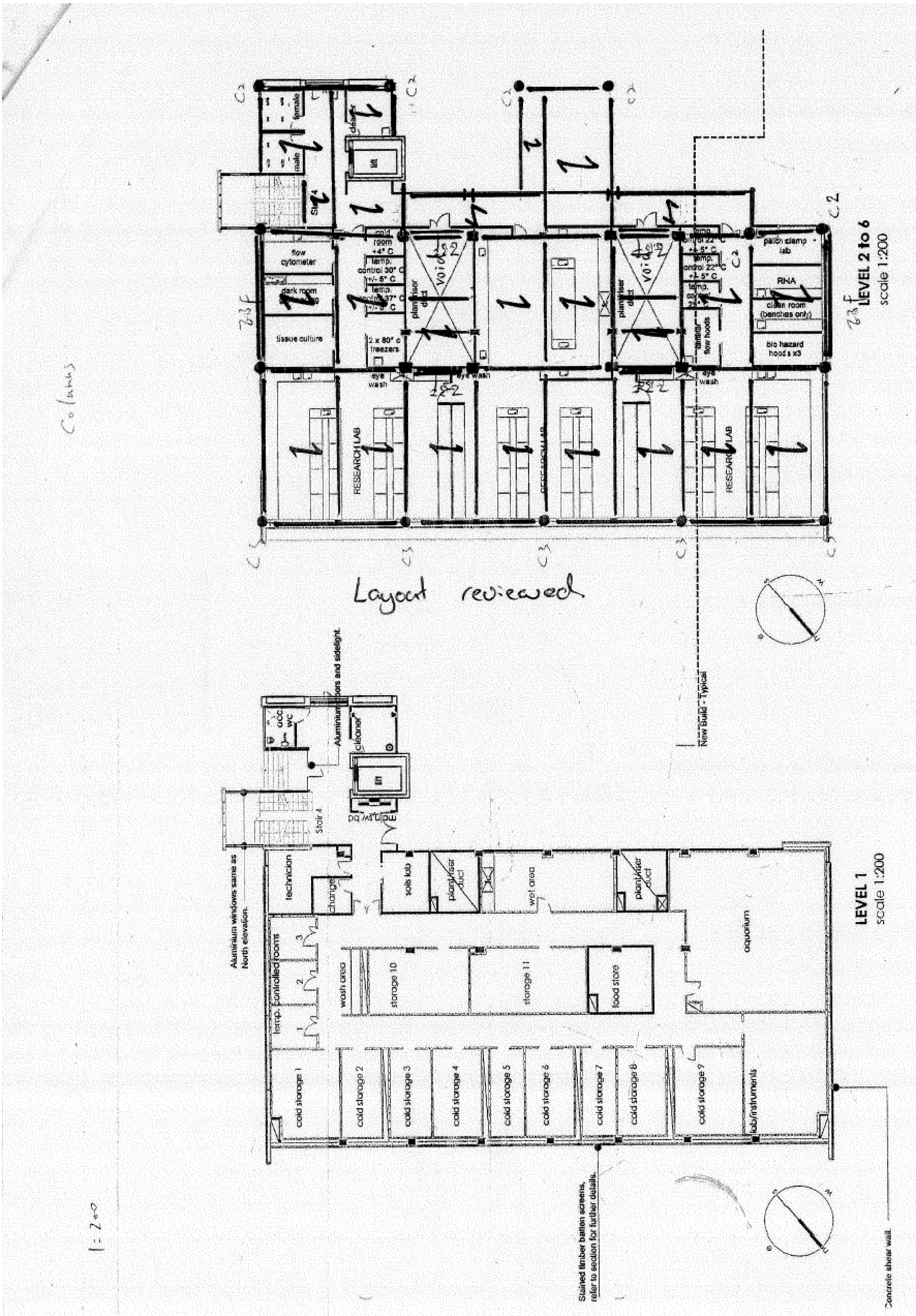

LEVEL 2 to 6
scale 1:200
Layout reviewed
LEVEL 1
scale 1:200
RESEARCH LAB
aquarium
food store
wet area
soils lab
technician
lift
cleaner
New Build - Typical
Stained timber batten screens, refer to section for further details.
Aluminium windows same as North elevation.

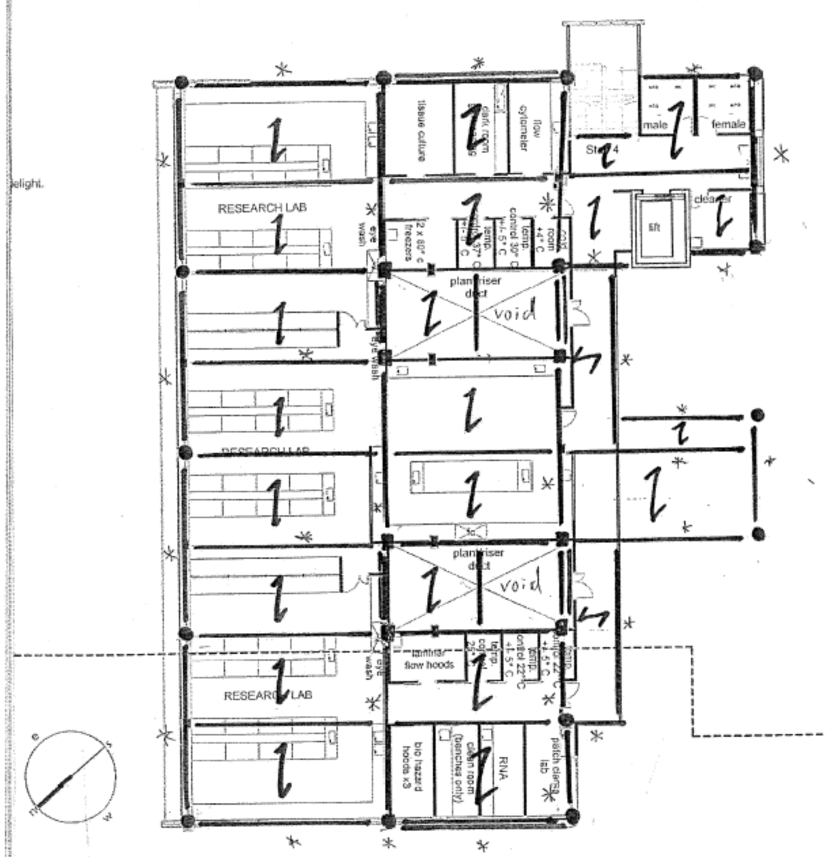

Columns
1:200
Layout reviewed
LEVEL 2 to 6
scale 1:200
LEVEL 1
scale 1:200
RESEARCH LAB
flow cytometer
dark room
tissue culture
cold room +4° C
2 x 80° c freezers
eye wash
plant/riser duct
void
patch clamp lab
RNA
clean room (benches only)
bio hazard hood s x3
laminar flow hoods
lift
female
male
cleaner
New Build - Typical
Aluminium windows same as North elevation.
Aluminium doors and sidelight.
technician
change
soils lab
wet area
aquarium
temp. controlled rooms
wash area
storage 10
storage 11
food store
cold storage 1
cold storage 2
cold storage 3
cold storage 4
cold storage 5
cold storage 6
cold storage 7
cold storage 8
cold storage 9
lab/instruments
Stair 4
acc wc
Stained timber batten screens, refer to section for further details.
Concrete shear wall

Notes.

1. Fire rating.
   - columns 60 min linings.
   - beams marked (*) 60 min sprayed on cementitious coating.
2. No painting to steel in internal air conditioned environment
3. Concrete strength f'c = 25 MPa.
4. 665 mesh to slab.
5. HD12 @ 300 c/c 1.5 m long saddle bars.
   HD12 @ 300 starter bars to perimeter of floor + openings.

I 150.

750

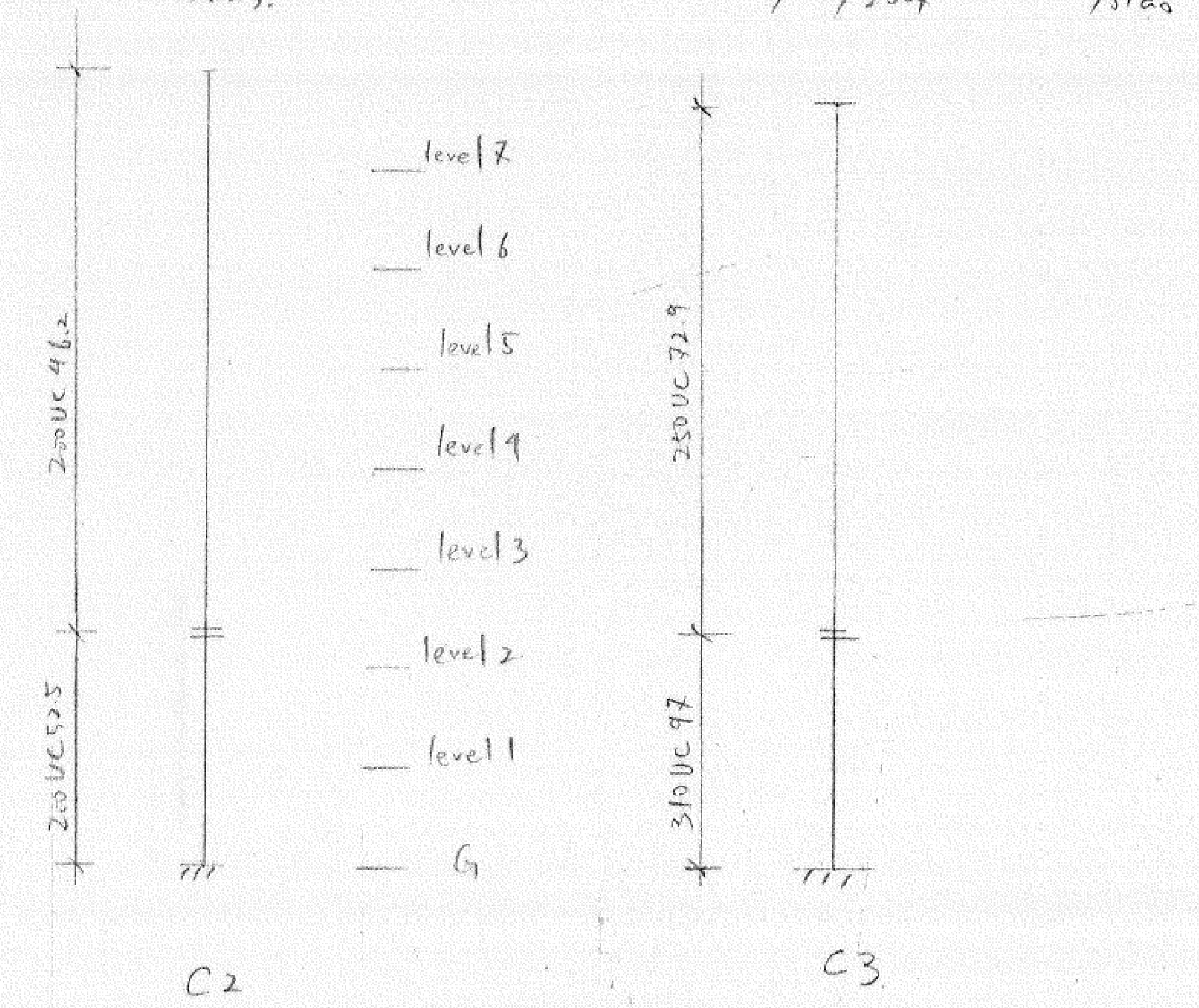


LEVEL 2 to 6
scale 1:200

Steel Structures Analysis Service

Project name Canterbury RS Building. | Project no. 73 | Page 15

Subject Columns. | Date 24/05/2007 | By Xiao



Option 1.

Floor Level 1, Thermomass wall (31mm thick).

ID	Designation	Studs #	Precamber?	Propped?	Connection
B1	410UB60	38	Yes		WP30
B1a	610UB101	30		Yes	WP30
B1b	410UB60	34		Yes	WP30
B2	360UB45	28	Yes		WP50
B2a	360UB45	28	Yes		WP50
B2b	360UB45	24			WP30
B2c	410UB54	32		Yes	WP30
B3	530UB82	52		Yes	WP50
B3a	610UB101	26		Yes	WP30
B3b	530UB82	50			WP50
B3c	610UB101	24			WP30
B4	310UB46	14			MEP50
B4a	310UB46	20			WP30
B5	200UB22	14			WP30
B5a	200UB22	14			WP30
B5b	200UB22	12			WP30
B6	200UB18	12			WP30
B7	250UB26	16			WP30
B8					

Option 2.

Typical floor 2 - 6, roof similar + light timber thermal wall at level 1

ID	Designation	Studs #	Precamber?	Propped?	Connection
B1	410UB60	38	Yes		WP30
B1a	410UB60	32		Yes	WP30
B1b	410UB60	34		Yes	WP30
B2	360UB45	28	Yes		WP50
B2a	360UB45	28	Yes		WP50
B2b	360UB45	24			WP30
B2c	410UB54	32		Yes	WP30
B3	530UB82	52		Yes	WP50
B3a	530UB82	24		Yes	WP30
B3b	530UB82	50			WP50
B3c	530UB82	24			WP30
B4	310UB46	14			MEP50
B4a	310UB46	20			WP30
B5	200UB22	14			WP30
B5a	200UB22	14			WP30
B5b	200UB22	12			WP30
B6	200UB18	12			WP30
B7	250UB26	16			WP30
B8	530UB92.4	44			WP30

Appendix B: Holmes Consultants Steel Design Review

HolmesConsultingGroup

CORRESPONDENCE

STRUCTURAL AND CIVIL ENGINEERS

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29 May 2008

Dr Andrew Buchanan
University of Canterbury
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Dear Andy

FRST TIMBER RESEARCH - BIOLOGICAL SCIENCES STEEL OPTION REVIEW

As requested, we have been through the preliminary steel design options prepared by Steel Construction NZ. We can comment as follows:-

General

The design as presented is reasonably comprehensive for a preliminary design study, and appears to address most of the main structural elements, with the exception of the roof steelwork and the foundations. That said, it is generally of a similar level to the timber alternative that we were given, so is a useful point of comparison for a preliminary estimate.

Design Issues

There are a few areas of difference that may need further consideration:-

1. The foundation design will be sufficiently different that it ought to be factored into the comparison. The timber walls are generally on the line of the basement walls which therefore provide a significant rigid foundation beam effect, spreading load to the limits of the basement. The longitudinal frames appear to be most effective on line A, and therefore immediately over the basement, while we would expect the frame on line B to take less of the seismic load. This results in a relatively modest increase in loads to the basement floor.

   Conversely the steel building is using braced frames on or near lines B and C. Single bay braced frames such as these result in large seismic axial loads to the columns and hence the potential for significantly larger foundation loads than would result from a moment frame. It is likely that there will be a need for tension piles or ground anchors with the braced frames as shown.

2. The positioning of the central braced frames is also a problem. The steel plans indicate the frames on the line of the edge of the void, which results in the frames emerging in the storage spaces at Level 1. Addressing this would require a change to the architectural layout, or the braced frames need to relocate back onto line B. This would increase the size of the steel secondary members significantly, resulting in an overall increase in the weight of steel. Moreover, the braced frames on lines 1 and 9 would reduce in length, resulting in increased flexibility and heavier axial elements.

   The accompanying architectural drawings show the steel frames on Grid B, meaning that the frames will not be as presented in the sketches.

   This issue has been addressed in the timber option, which has the structure on Grid B as planned.

3. With such a reduced number of frames supporting the overall seismic load, we believe that the building may be excessively flexible. On the basis of a quick calculation, we determined an overall displacement in excess of 350mm in the worst case. Although the interstorey drifts may be within acceptable code limits, there may be significant practical issues in dealing with these displacements in the detailing of other elements.

   Normally, we would expect to see longer bracing elements in a building of this size, probably in multiple bay frames, resulting in stiffer structure with less significant uplift loads under the columns.

4. Although we have not performed any analysis, vibration of the floor should also be considered. In the case of potentially sensitive areas such as laboratory floors, lighter structural steel floors are potentially a problem (although we are not sure how this would compare for the timber option).

5. The floors are a combination of propped and unpropped construction. This is not necessarily an issue, but it should be noted that there is a time cost in dealing with this, and so some of the benefit of steel is negated by this.

6. No mention is made of the reinforcing in the concrete floor. We would normally expect to see a combination of 'ductile' mesh and/or mild steel, to provide shrinkage and thermal reinforcing, seismic diaphragm reinforcing and supplementary fire reinforcing. Given the spans of the Comfloor, the latter may govern. (Note a plan of this was referred to, but we do not have it).

7. Although the Engineer points out that the secondary beams may not require fire treatment if the slab panel method of analysis were used, this implies possible severe damage in the event of fire (even though life safety would be achieved in accordance with the Building Code). This could be a performance issue that would

be addressed with the client normally, possibly with a recommendation that fire treatment be installed to all members, even if only a boarded system.

With the notes above, we are generally satisfied that the steel design is a reasonable preliminary comparative design. The first two items are probably the most significant, and it may be suitable to simply allow a margin on the foundation design of say, 15%, in order to restore some relativity. This can only be verified by more intensive design.

Please contact the undersigned if you have further questions in regard.

Yours sincerely

John Hare
DIRECTOR

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